This is a digital copy of a book that was preserved for generations on library shelves before it was carefully scanned by Google as part of a project to make the world's books discoverable online.

It has survived long enough for the copyright to expire and the book to enter the public domain. A public domain book is one that was never subject to copyright or whose legal copyright term has expired. Whether a book is in the public domain may vary country to country. Public domain books are our gateways to the past, representing a wealth of history, culture and knowledge that's often difficult to discover.

Marks, notations and other marginalia present in the original volume will appear in this file - a reminder of this book's long journey from the publisher to a library and finally to you.

Usage guidelines

Google is proud to partner with libraries to digitize public domain materials and make them widely accessible. Public domain books belong to the public and we are merely their custodians. Nevertheless, this work is expensive, so in order to keep providing this resource, we have taken steps to prevent abuse by commercial parties, including placing technical restrictions on automated querying.

We also ask that you:

+ *Make non-commercial use of the files* We designed Google Book Search for use by individuals, and we request that you use these files for personal, non-commercial purposes.

+ *Refrain from automated querying* Do not send automated queries of any sort to Google's system: If you are conducting research on machine translation, optical character recognition or other areas where access to a large amount of text is helpful, please contact us. We encourage the use of public domain materials for these purposes and may be able to help.

+ *Maintain attribution* The Google "watermark" you see on each file is essential for informing people about this project and helping them find additional materials through Google Book Search. Please do not remove it.

+ *Keep it legal* Whatever your use, remember that you are responsible for ensuring that what you are doing is legal. Do not assume that just because we believe a book is in the public domain for users in the United States, that the work is also in the public domain for users in other countries. Whether a book is still in copyright varies from country to country, and we can't offer guidance on whether any specific use of any specific book is allowed. Please do not assume that a book's appearance in Google Book Search means it can be used in any manner anywhere in the world. Copyright infringement liability can be quite severe.

About Google Book Search

Google's mission is to organize the world's information and to make it universally accessible and useful. Google Book Search helps readers discover the world's books while helping authors and publishers reach new audiences. You can search through the full text of this book on the web at `http://books.google.com/`

WIDENER LIBRARY
HX HH5H K

Sci 1520.383

Bd. Oct. 1890.

Harvard College Library

FROM THE BEQUEST OF

MRS. ANNE E. P. SEVER,

OF BOSTON,

WIDOW OF COL. JAMES WARREN SEVER,

(Class of 1817),

22 April - 10 Sept. 1890.

SCIENCE CENTER LIBRARY

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

HARVARD COLLEGE
APR 22 1890
LIBRARY.

JANUARY,

1890.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Library.

CONTENTS.

TRANSACTIONS. VOL. XXII.

No.		Page
431.	The Results of Investigations Relative to Formulas for the Flow of Water in Pipes. EDMUND B. WESTON	1
....	Discussion— By RUDOLPH HERING, CHARLES E. EMERY, E. SHERMAN GOULD, MANSFIELD MERRIMAN, JOHN R. FREEMAN and EDMUND B. WESTON	66

ILLUSTRATIONS.

I—XXVIII. Diagrams Illustrating Paper No. 431 64

The Transactions of the Society will be sent to any subscriber at the rate of $10 per year; and to clubs of ten or more, when ordered through the Secretary of an Engineering or Technical Society or Club, who will be responsible for the payment, at 25 per cent. discount.

PUBLISHED AT THE HOUSE OF THE SOCIETY, 127 EAST TWENTY-THIRD STREET, NEW YORK, ON THE SECOND WEDNESDAY OF THE MONTH.

Entered according to Act of Congress, in the year 1890, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Digitized by Google

American Society of Civil Engineers.

OFFICERS FOR 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN, WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Standing Committees.

On FINANCE :—Charles B. Brush, Mendes Cohen, Robert Van Buren.
On LIBRARY :—Alphonse Fteley, Theodore Voorhees, William Ludlow.

Special Committees.

THE PRESIDENT OF THE SOCIETY IS HONORARY MEMBER OF ALL COMMITTEES.

ON UNIFORM STANDARD TIME :—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston, Fred. Brooks.

ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY :—F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, A. V. Abbott.

ON UNIFORM METHODS OF TESTING MATERIALS USED IN METALLIC STRUCTURES, AND ON REQUIREMENTS FOR THESE MATERIALS TO FURTHER IMPROVE THE GRADE OF SUCH STRUCTURES:—G. Bouscaren, William H. Burr, J. G. Dagron, R. W. Hunt, H. G. Morse, P. Roberts, Jr., H. B. Seaman.

MEETINGS.

Regular Meetings, on the first and third Wednesdays of the month, at 20 o'clock, except during July and August.

The rooms of the Society are open from 9 to 17 o'clock each business day, except Saturday, when they are closed at 15 o'clock.

The Library and Conversation Room will also, for the present, be open every *Wednesday Evening*, from 19½ to 22 o'clock. Members are invited to avail themselves of the opportunities afforded on these evenings, both for consultation of books and periodicals, and for conversation.

HOUSE OF THE SOCIETY—127 EAST TWENTY-THIRD STREET, NEW YORK.

Digitized by Google

TA 5

TRANSACTIONS

053.2-5

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852.)

VOL. XXII.

JANUARY TO JUNE, 1890.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1890.

Digitized by Google

Sci 1520.388

VIII. 570

OFFICERS FOR THE YEAR ENDING DECEMBER 31, 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN,
WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Entered according to Act of Congress, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the office of the Librarian of Congress at Washington.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

Digitized by Google

CONTENTS.

PAPERS.

NO.	MONTH.		PAGE.
432	(*January.*)	The Results of Investigations Relative to Formulas for the Flow of Water in Pipes.	
		EDMUND B. WESTON	1
....	(*do.*)	Discussion on the Results of Investigations Relative to Formulas for the Flow of Water in Pipes:	
		By RUDOLPH HERING	66
		By CHARLES E. EMERY	69
		By E. SHERMAN GOULD	71
		By MANSFIELD MERRIMAN	72
		By JOHN R. FREEMAN	74
		By EDMUND B. WESTON	68, 70, 72, 73, 83
432	(*February.*)	Cast Iron—Strength, Resilience, Tests and Specifications.	
		J. B. JOHNSON	91
....	(*do.*)	Discussion on Cast Iron:	
		By GEORGE A. JUST	121
		By LOUIS DE COPPET BERG	128
		By J. B. JOHNSON	129
433	(*March.*)	The Improvement of Railway and Street Railway Track.	
		E. RUSSELL TRATMAN	135
434	(*April.*)	Development of the American Rail and Track.	
		J. ELFRETH WATKINS	209
435	(*do.*)	Railways of Mexico.	
		WILLIAM BARCLAY PARSONS	233
436	(*do.*)	The Railroad Ferry Steamer "Solano."	
		ROBERT L. HARRIS	247
437	(*May.*)	A New Graphical Solution of the Problem, What Position a Train of Concentrated Loads must have in order to cause the Greatest Stress in any Given Part of a Bridge Truss or Girder.	
		HENRY T. EDDY	259
438	(*June.*)	Address at the Annual Convention at Cresson, Pa., June 26th, 1890.	
		WILLIAM P. SHINN	359
439	(*do.*)	A Method of taking Cross-Sections in Deep Rock Cuts by Triangulation.	
		F. W. WATKINS	386
440	(*do.*)	Calculation of the Mean Horse Power of a Variable Stream, and the Cost of Replacing the Power Lost by a Partial Diversion of the Flow.	
		WILLIAM H. GRANT	389

Digitized by Google

NO.	MONTH.		PAGE.
....	(*June.*)	Discussion on Mean Horse Power of Stream:	
		By CHARLES E. EMERY	402, 403, 404
		By WILLIAM E. WORTHEN	403, 404
		By EDWARD P. NORTH	403
		By JOSEPH P. DAVIS	404
		By WILLIAM H. GRANT	403, 404, 405
441	(*do.*)	Observations on the Forth Bridge.	
		CHARLES E. EMERY	409

ILLUSTRATIONS.

PLATE.	MONTH.		PAPER.	PAGE.
I-XXVIII.	(*January.*)	Plates illustrating Paper 431 on the Results of Investigations Relative to Formulas for the Flow of Water in Pipes	431	64
XXIX.	(*February.*)	Tension Tests of Cast Iron	432	98
XXX.	(*do.*)	Strain Diagrams, Foundry Cast Iron	432	100
XXXI.	(*do.*)	Strain Diagrams, Gun Iron	432	101
XXXII.	(*do.*)	Level Micrometer for Measuring Extension	432	103
XXXIII.	(*do.*)	Heisler's Impact Testing Machine	432	111
XXXIV.	(*do.*)	Diagram showing absence of any relation between Strength and Resilience of Cast Iron	432	120
XXXV.	(*do.*)	Photographs of Specimens Tested	432	124
XXXVI.	(*do.*)	Photographs of Lintels Tested	432	128
XXXVII.	(*March.*)	Rail Sections, Joints and Bolts	433	208
XXXVIII.	(*do.*)	Rail Sections, Tie Plates, Joints and Braces	433	208
XXXIX.	(*do.*)	Tie Plates, Fastenings, Joints, Bolts, Sections of Road Beds, Bridge Floors	433	208
XL.	(*do.*)	Effect of Joints on Bolts	433	208
XLI.	(*do.*)	Street Railway Rails and Sub-structure	433	208
XLII.	(*April*)	Facsimile of Letter of Robert L. Stevens	434	209
XLIII-XLVI	(*do.*)	Cast Iron Rails used from 1789 to 1816	434	216
XLVII.	(*do.*)	Early English Rolled Rails	434	218
XLVIII.	(*do.*)	Section of Rail, Portage Railroad of Pennsylvania, 1833.	434	218
XLIX-L.	(*do.*)	Stevens' Rail, Camden and Amboy Railroad, 1837	434	222
LI.	(*do.*)	Strap Rail, Albany and Schenectady Railroad, 1837	434	222
LII.	(*do.*)	Strap Rail, Baltimore and Ohio Railroad, 1837	434	222
LIII.	(*do.*)	Sections of Rails	434	228
LIV.	(*do.*)	Angle Splice Bar, Erie Railroad, 1857	434	230
LV.	(*do.*)	Frogs and switches used on Colliery Roads of England, 1825	434	232
LVI.	(*do.*)	Ferry Steamer "Solano" in Dock	436	258
LVII.	(*do.*)	Photograph of Steamer	436	258
LVIII.	(*do.*)	Map showing Straits of Carquinez	436	258
LIX.	(*do.*)	Cross-section and Elevation of Boat and Apron	436	258
LX.	(*do.*)	Plan of Boat	436	258
LXI.	(*do.*)	Plate showing Details of Construction	436	258
LXII.	(*do.*)	Plan of Boat and Steering Gear	436	258
LXIII.	(*do.*)	Arrangement for Burning Oil in Boilers	436	258
LXIV.	(*June.*)	Cross-section, West Approach Cut, Weehawken	439	388
LXV.	(*do.*)	Geometrical Solution of Triangulation	439	388
LXVI.	*do.*)	Diagram A, illustrating the mean yearly Flow and Horse-power of the Bronx River	440	408

Digitized by Google

PLATE.	MONTH.		PAPER.	PAGE.
LXVII.	(*June.*)	Diagram B, illustrating the Mean Yearly Flow and Horse-power of the Bronx River after the Diversion of 45 per cent. of the Normal Flow.....	440	408
LXVIII.	(*do.*)	The Forth Bridge, Elevation and Sections.....	441	424
LXIX.	(*do.*)	Diagrams showing Temperature Movements.....	441	424

ERRATA.

Transactions, American Society of Civil Engineers, Vol. XXII:

February, 1890, page 108, 15th line from bottom, for $R = \frac{f^2}{30E}$ read $R = \frac{f^2}{24E}$; 12th line from bottom, for "15 times" read 12 times.

April, 1890, page 221, foot note, for "Stevens" read Stevenson's. Page 225, 12th line from bottom, for "Erie Railroad" read Pennsylvania Railroad. Page 230, 15th line from bottom, for "36 inches" read 48 inches.

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

HARVARD COLLEGE
APR 22 1890
LIBRARY

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

431.

(Vol. XXII.—January, 1890.)

THE RESULTS OF INVESTIGATIONS RELATIVE TO FORMULAS FOR THE FLOW OF WATER IN PIPES.

By EDMUND B. WESTON, M. Am. Soc. C. E.

WITH DISCUSSION.

INTRODUCTORY.

About thirteen years ago, when the writer first commenced to make a practical application of hydraulic formulas, he was unable to find one for calculating the loss of head due to the friction of water flowing in pipes, which he had not heard criticised more or less unfavorably by hydraulic engineers.

This unsatisfactory state of affairs led him to commence to make special investigations upon the subject, and to collect original data of an experimental nature relating to the same, in order to prove to his own satisfaction, if possible, if any of the formulas that he was familiar with were reliable for general use, and if not, to endeavor to construct one.

The results of these researches, which are illustrated by sketches and diagrams, are now presented in this paper.

The data of five hundred and twenty experiments were obtained, which are tabulated in a systematic manner in Table No. 1.

A careful study of the experimental data convinced the writer that the same formula would not apply to all cases, and that in order to make

Digitized by Google

an intelligent investigation, it would be necessary to divide it into the three following classes, viz.: that which had been obtained with pipes having very smooth interior sides similar to brass and lead pipes, that which had been obtained with pipes having interior sides similar to new cast-iron pipes, and that which had been obtained with pipes having interior sides similar to old cast-iron pipes whose interior sides had become roughened by oxidation.

The result of this division was the construction by the writer of a new formula for the flow of water in pipes having very smooth interior sides, and his coming to the conclusion that two formulas constructed by the eminent French Civil Engineer, Henry Darcy, were very well adapted for pipes having interior sides similar to new cast-iron pipes.

A general formula was not found that would satisfactorily apply to old cast-iron pipes having interior sides that had become roughened by oxidation, and it was not possible to construct one from the limited data at hand, although a formula by Darcy, and several by the writer constructed for individual cases, agree very well with the experimental results from which they were derived.

The writer also briefly discusses the form of formulas the best adapted for the flow of water in pipes, and calls attention to formulas for computing the co-efficients of resistance, for enlargements, contractions, elbows and curves.

The investigations and subjects that this paper treats upon will hereafter be referred to, or described in detail in the following order:

First.—The Nomenclature. (Page 3.)
Second.—The form of Formulas best adapted for the Flow of Water in Pipes. (Page 4.)
Third.—The Co-efficient of Influx. (Page 7.)
Fourth.—Description of Table No. 1. (Page 8.)
Fifth.—Table No. 1. (Page 11.)
Sixth.—Description of the apparatus used in making the experiments, the results of which are contained in Table No. 1. (Page 23.)
Seventh.—A Formula for the Flow of Water in Pipes having very smooth Interior Sides. (Page 48.)
Eighth.—Formulas for the Flow of Water in Pipes having Interior Sides similar to New Cast-Iron Pipes. (Page 56.)
Ninth.—Old Cast-Iron Pipes Lined with Deposit, and the same cleaned. (Page 60.)
Tenth.—The Co-efficients of Resistance to the Flow of Water in Pipes, for Enlargements, Contractions, Elbows and Curves. (Page 61.)

Digitized by Google

NOMENCLATURE.

Unless distinctly stated to the contrary, the following characters will always represent English feet when they apply to linear distances, and have these significations:

h = the head. When the pipe is submerged, it is the difference in elevation between the surface of the water at the inlet and outlet; or when the pipe discharges into the open air, it is the difference in elevation between the surface of the water at the inlet, and the center of the outlet end of the pipe.

h_f = the loss of head due to friction, of the water flowing against the interior sides of the pipe, etc.

l = the length of the pipe.

d = the mean interior diameter of the pipe.

g = the acceleration of gravity.

v = the mean velocity, per second, of the water flowing in the pipe, or the velocity, per second, of the water issuing from the outlet end of the pipe.

ε = the co-efficient of influx or resistance of the entrance of the water into the pipe.

Δ = the co-efficient of contraction of the entrance of the water into a pipe.

ζ = the co-efficient of friction of the water flowing against the interior sides of the pipe.

ϕ = the co-efficient of resistance of the water passing from a large to a small pipe.

μ = the co-efficient of resistance of the water passing from a small to a large pipe.

F = the area or cross section of a pipe.

ψ = the co-efficient of resistance of the water flowing in a curved pipe.

r = radius of curvature of a curved pipe.

τ = the co-efficient of resistance of the water flowing in an elbow pipe.

ρ = angle of an elbow pipe.

m = the total number of discharge pipes that are supplied by the single pipe under consideration.

α = an experimental derivation.

β = an experimental derivation.

z = a diametric co-efficient.

J = the sine of the inclination of the water surface, or fall in a length of 1 (used only in Kutter's formula).

n = the co-efficient of roughness dependent upon the surface of the material over which the water flows (used only in Kutter's formula).

ω = a variable co-efficient.

b =
c =
e =
k =
o =
p =
x =
y = } characters of Figures Nos. 1 and 2.

Digitized by Google

THE FORM OF FORMULAS BEST ADAPTED FOR THE FLOW OF WATER IN PIPES.

The experience of quite a number of years in solving hydraulic problems, has led the writer to the conclusion that the forms of formulas most convenient for ascertaining the different results generally sought after, relating to the flow of water in straight pipes, are the following:

$$h_f = \zeta \frac{l}{d} \frac{v^2}{2g}.$$

$$v = \frac{\sqrt{2gh_f}}{\sqrt{\zeta \frac{l}{d}}}.$$

$$h = \frac{v^2}{2g} + \varepsilon \frac{v^2}{2g} + \zeta \frac{l}{d} \frac{v^2}{2g} = \left(1 + \varepsilon + \zeta \frac{l}{d}\right) \frac{v^2}{2g}.$$

$$v = \frac{\sqrt{2gh}}{\sqrt{1 + \varepsilon + \zeta \frac{l}{d}}}.$$

It is assumed in these forms of formulas that the loss of head due to friction increases at the same time with the square of the velocity (v^2), and with conditions that are included in the co-efficient of friction ζ, of the water flowing in the pipes. This co-efficient of friction, which is an experimental derivation, is not expressed individually in the majority of the formulas that have been published by different authorities who have made investigations upon the flow of water in pipes, nor is its value the same in these formulas, nor the laws upon which it is based. For instance, among the formulas that the writer is conversant with, in addition to being dependent upon the nature of the interior sides of the pipes, there are several in which it is dependent upon the velocity (v), one in which it is dependent upon the square root of the velocity ($\sqrt{v}$), one in which it is dependent upon the diameter (d) and one in which it is dependent upon both the velocity and diameter, etc.

The following are several of the different forms of formulas that are often met with in hydraulic text books :

$$h_f = \left(\alpha v + \beta v^2\right) \frac{l}{d}.$$

$$h_f = \frac{l}{\omega^2} \frac{v^2}{d}.$$

$$v = \omega \sqrt{\frac{h_f d}{l}}.$$

Digitized by Google

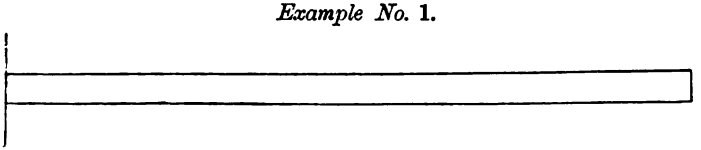
These forms of formulas are undoubtedly a little more convenient for solving very simple problems relating to straight pipes, than are those recommended by the writer; but they cannot be utilized without a considerable modification for solving the more complicated problems that sometimes arise in practice, such as the flow of water from a reservoir, through a straight pipe, a compound pipe, or a compound system of pipes; whereas the forms of formulas, recommended by the writer, can each be readily extended to cover a great variety of cases of this kind, as they contain all of the elements plainly set forth that are necessary for calculating the flow of water in pipes, with the exception of a little supplementary data, such as the co-efficients of resistance for contraction, enlargement, elbows and curves, which can easily be obtained and introduced when necessary. Then there is always a great advantage in having before you individually the co-efficient of friction ζ, which is practically the only important element in a formula, concerning the pipes themselves, that is determined by experiment, and upon which almost entirely depends the conversion of the theoretical head or velocity into the actual.

The co-efficient of friction ζ, as expressed in the formulas recommended by the writer, has been adopted for the reduction of all the experimental data contained in this paper, as well as for the basis of all formulas that have been constructed and all comparisons that have been made, relative to the flow of water in pipes.

The following equations illustrate the manner in which the formulas recommended can be extended to cover a number of cases. For facility of comprehension, the examples to which the different equations apply, are expressed by sketches (without regard to scale).

These sketches are intended to represent plans of different pipes, and combinations of pipes, all of which are supposed to be located below the hydraulic grade line.

Example No. 1.

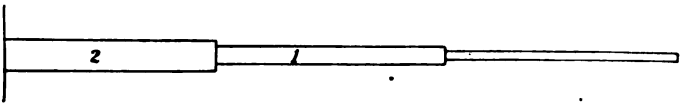


$$v = \frac{\sqrt{2gh}}{\sqrt{1 + \varepsilon + \zeta \frac{l}{d}}}$$

$$h_f = \left(\varepsilon + \zeta \frac{l}{d} \right) \frac{v^2}{2g}.$$

Digitized by Google

Example No. 2.



$$v = \frac{\sqrt{2gh}}{\sqrt{1+\left(\varepsilon+\zeta_2\frac{l_2}{d_2}\right)\left(\frac{d}{d_2}\right)^4+\left(\phi_1+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4+\phi+\zeta\frac{l}{d}}}.$$

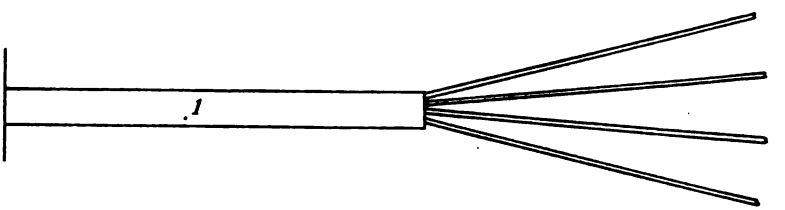
$$h_f=\left[\left(\varepsilon+\zeta_2\frac{l_2}{d_2}\right)\left(\frac{d}{d_2}\right)^4+\left(\phi_1+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4+\phi+\zeta\frac{l}{d}\right]\frac{v^2}{2g}.$$

l, d, ζ, ϕ and v refer to the discharge pipe.

l_1, d_1, ϕ_1 and ζ_1 refer to pipe 1.

l_2, d_2, ε and ζ_2 refer to pipe 2.

Example No. 3.



$$v = \frac{\sqrt{2gh}}{\sqrt{1+\left(\varepsilon+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4 m^2+\phi+\zeta\frac{l}{d}}}.$$

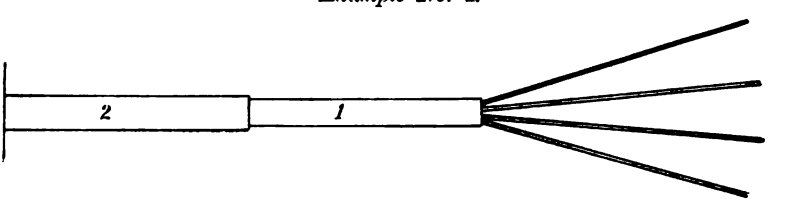
$$h_f=\left[\left(\varepsilon+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4 m^2+\phi+\zeta\frac{l}{d}\right]\frac{v^2}{2g}.$$

m = the total number of small discharge pipes.

l, d, ϕ, ζ and v refer to one of the small discharge pipes, their dimensions all being the same.

l_1, d_1, ε and ζ_1 refer to pipe 1.

Example No. 4.



Digitized by Google

$$v = \frac{\sqrt{2gh}}{\sqrt{1+\left(\varepsilon+\zeta_2\frac{l_2}{d_2}\right)\left(\frac{d}{d_2}\right)^4 m^2+\left(\phi_1+\zeta_1\frac{l_1}{d_1}\left(\frac{d}{d_1}\right)^4 m^2+\phi+\zeta\frac{l}{d}\right.}}$$

$$h_f = \left[\left(\varepsilon+\zeta_2\frac{l_2}{d_2}\right)\left(\frac{d}{d_2}\right)^4 m^2+\left(\phi_1+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4 m^2+\phi+\zeta\frac{l}{d}\right]\frac{v^2}{2g}$$

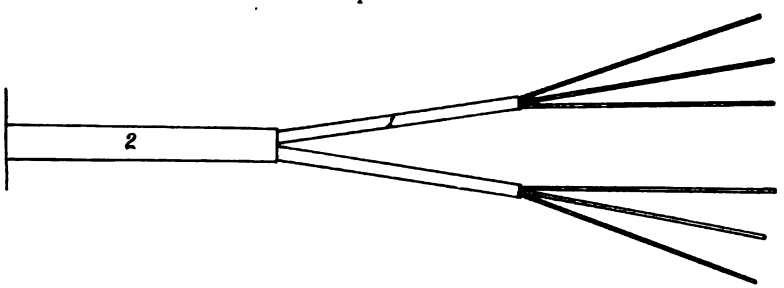
m = the total number of small discharge pipes.

l, d, ϕ, ζ and v refer to one of the small discharge pipes, their dimensions all being the same.

l_1, d_1, ϕ_1 and ζ_1 refer to pipe 1.

l_2, d_2, ε and ζ_2 refer to pipe 2.

Example No 5.



$$v = \frac{\sqrt{2gh}}{\sqrt{1+\left(\varepsilon+\zeta_2\frac{l_2}{d_2}\right)\left(\frac{d}{d_2}\right)^4 m_1^2 m^2+\left(\phi_1+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4 m^2+\phi+\zeta\frac{l}{d}}}$$

$$h_f = \left[\left(\varepsilon+\zeta_2\frac{l_2}{d_2}\right)\left(\frac{d}{d_2}\right)^4 m_1^2 m^2+\left(\phi_1+\zeta_1\frac{l_1}{d_1}\right)\left(\frac{d}{d_1}\right)^4 m^2+\phi+\zeta\frac{l}{d}\right]\frac{v^2}{2g}$$

m = the number of small discharge pipes that are connected with one of the branch pipes.

m_1 = the number of branch pipes.

l, d, ϕ, ζ and v refer to one of the small discharge pipes, their dimensions all being the same.

l_1, d_1, ϕ_1 and ζ_1 refer to one of the branch pipes, the dimensions of both being the same.

l_2, d_2, ε and ζ_2 refer to pipe 2.

THE CO-EFFICIENT OF INFLUX.

As a co-efficient of influx, or of resistance, ε, to the entrance of the water into the inlet ends of cylindrical pipes, when the edges of these pipes are square and flush with the face of a wall or partition, was fre-

Digitized by Google

quently used in computing the experimental co-efficients of friction (ζ), contained in Table No. 1, the writer will make a few remarks concerning it before proceeding to describe the table, viz.:

$$\varepsilon = \left(\frac{1}{d}\right)^2 - 1 \text{ and } h_f = \varepsilon \frac{v^2}{2g}.$$

0.505 is recommended by Weisbach, which is a result deduced from experiments made by himself.
0.513 is derived from the results of experiments made by Michelotte with tubes from 0.5 to 3 inches in diameter, under a head of water varying from 3 to 20 feet.
0.456 is derived from the results of five experiments made by Castel with tubes 0.61 inches in diameter.
0.539 is derived from the results of five experiments made by Bossut with tubes from 0.91 to 1.03 inches in diameter.
0.484 is derived from the result of an experiment made by Eytelwein with a tube 1.02 inches in diameter.
0.481 is derived from the result of an experiment made by Venturi with a tube 1.61 inches in diameter.
0.469 is derived from the results of approximate experiments made by Darcy with pipes from 1.42 to 11.81 inches in diameter.

The writer considers the co-efficient 0.505, recommended by Professor Weisbach, the most reliable. Therefore whenever the co-efficient of influx ε has been used in this paper, its value was considered as 0.505, unless otherwise distinctly specified.

DESCRIPTION OF TABLE No. 1.

This table contains the results of five hundred and twenty experiments, which have been made to ascertain the laws relating to the flow of water in pipes.

In making the experiments fifty-seven different sizes of pipe were used, ranging from 0.409 to 90 inches in diameter, and from 12 feet to 60,000 feet in length, which were constructed of tin, zinc, brass, glass, lead, sheet-iron, artificially coated and not coated, wrought-iron, new and old, cast-iron, new and old, artificially coated and not coated, wood, earthenware and brick.

The first column contains the tabular numbers of the experimental results.

The second column contains the mean diameter of the interior of the pipes.

The third column contains the heads, which are in all cases the total heads, unless they are indicated by *, when they are the frictional heads, due to the water flowing against the interior sides of the pipes.

The fourth column contains the lengths of the pipes. When the total head is given in the third column, opposite to it, in this column, is given the entire distance from the inlet to the outlet of the pipe; this

Digitized by Google

is also the case if the frictional head is given, provided the loss of head due to the entrance of the water into the pipe, and the head required to generate the velocity, have simply been subtracted from the total head; but if manometric tubes, or substitutes for the same, were used in determining the head, the length given is the distance between the points where the tubes were connected to the pipe.

The fifth column contains the mean velocity of the water that was flowing in the pipes.

The sixth column contains the co-efficients of influx, or of resistance, to the entrance of the water into the pipes, if any were required, that were used in computing the co-efficients of friction (ζ), given opposite in the seventh column. When the frictional head is given in the third column, the co-efficient of influx does not enter into the problem. If, on the contrary, the total head is given, a co-efficient of influx was necessarily always employed, if the pipes did not have funnel shaped inlets, and in cases where one had not been determined by experimental investigation, 0.505 was used for reasons that have already been explained. Consequently if 0.505 is given in the sixth column, it is to be understood that it is an assumed value, and when other values are given it implies that they have been determined by experiment to meet the requirements of their respective cases.

The seventh column contains the co-efficients of friction (ζ), which have been computed from the results of the experiments contained in the table. When the frictional head was used, the equation employed for the determination of the co-efficient of friction (ζ), was

$$\zeta = \frac{2g\, h_f}{v^2} \frac{d}{l};$$

but when the total head was used, the head required to generate the velocity and the loss of head due to the entrance of the water into the pipe, had to be considered, and when the co-efficient of influx was assumed to be 0.505, the equation employed for a case of this kind was

$$\zeta = \left(\frac{2g\, h}{v^2} - 1.505\right)\frac{d}{l}.$$

The values of $2g$, that were used in the computations, have ranged between 64.326 and 64.4, depending upon the locality where the experiments were made. All of the experimental co-efficients of friction (ζ) given in this column may be used in the equation,

$$h_f = \zeta \frac{l}{d} \frac{v^2}{2g},$$

and in the other equations previously mentioned and recommended by the writer; so that should an instance arise in which any of the formulas that will be given in this paper, in the future, do not apply, the table can be referred to, and quite possibly an independent experimental

Digitized by Google

co-efficient of friction (ζ) can be found, which will exactly fill the requirements of the case.

The eighth column contains the names of the authorities under whose direction the experiments were made or published, and the nature of the pipes that were used.

The ninth column contains the numbers of the diagrams upon which the co-efficients of friction (ζ) are platted that were used by the writer in determining his formula, and those upon which other co-efficients of friction (ζ) have been platted for comparison with formulas of different authorities.

There are quite a number of experimental results given in the table, which have not been platted on the diagrams nor mentioned in any form outside of the table, as they were not deemed sufficiently reliable, with the exception of those which were obtained from experiments that were made with wooden pipe, which were not required. These results can be ascertained by referring to the ninth column, as the space opposite to them in this column will not contain a diagram number.

Digitized by Google

TABLE No. 1.

EXPERIMENTAL VALUES of the Co-efficients of Friction (ζ) of Water in Pipes of Different Sizes, the Data from which they were Obtained, and the Number of the Diagram on which they are Platted.

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							WEISBACH.	
1	0.409	.....	.....	0.62	.....	0.0527	Brass pipe.....	1 and 7
2	"	.....	.....	0.63	.....	0.0676	Brass pipe.....	1 and 7
3	"	.....	.....	0.70	.....	0.0498	Brass pipe.....	1 and 7
4	"	.....	.....	1.87	.....	0.0365	Brass pipe.....	
5	"	.....	.....	6.63	.....	0.0273	Brass pipe.....	
6	"	.....	.....	27.92	.....	0.0182	Glass pipe.....	1 and 7
7	"	.....	.....	28.35	.....	0.0187	Brass pipe.....	1 and 7
8	"	.....	.....	40.42	.....	0.0178	Brass pipe.....	1 and 7
9	"	.....	.....	68.87	.....	0.0169	Brass pipe.....	1 and 7
10	0.563	.....	.....	0.73	.....	0.0607	Glass pipe.....	1 and 7
11	"	.....	.....	28.41	.....	0.0172	Brass pipe.....	1 and 7
12	"	.....	.....	33.40	.....	0.0187	Glass pipe.....	1 and 7
13	"	.....	....	40.68	.....	0.0174	Brass pipe.....	1 and 7
14	"	.....	.....	70.84	.....	0.0148	Brass pipe.....	1 and 7
							DARCY.	
15	0.480	* 0.28	328.09	0.11	.....	0.1719		25
16	"	0.60	"	0.23	.....	0.0869		25
17	"	1.00	"	0.38	.....	0.0582		25
18	"	1.75	"	0.48	.....	0.0590		25
19	"	2.47	"	0.55	.....	0.0632		25
20	"	5.44	"	0.75	.....	0.0751		25
21	"	8.47	"	0.94	.....	0.0750	New wrought-iron pipe..	25
22	"	11.39	"	1.13	.....	0.0706		25
23	"	14.43	"	1.29	.....	0.0685		25
24	"	20.55	"	1.57	.....	0.0656		25
25	"	28.07	"	1.88	.....	0.0624		25
26	"	58.60	"	2.78	.....	0.0597		25
27	"	112.95	"	3.92	.....	0.0577		25
							RENNIE.	
28	0.500	1.00	15.00	2.35	0.505	0.0283	Lead pipe.....	7
29	"	4.00	"	5.13	"	0.0230		7
							SMITH.	
30	0.502	0.82	11.13	2.08	0.488	0.0404		
31	"	1.27	"	2.72	"	0.0361	Glass pipe.....	
32	"	2.16	"	3.66	"	0.0335		
33	"	3.03	"	4.44	"	0.0316		
							DARCY.	
34	0.551	* 0.10	164.05	0.13	.....	0.1099		1
35	"	0.55	"	0.54	.....	0.0339		1 and 7
36	"	1.41	"	0.81	.....	0.0391		1 and 7
37	"	4.14	"	1.46	.....	0.0349	New lead pipe.....	1 and 7
38	"	10.08	"	2.40	.....	0.0315		1 and 7
39	"	18.76	"	3.44	.....	0.0286		1 and 7
40	"	26.49	"	4.23	.....	0.0267		1 and 7

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1—*(Continued.)*

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	$\frac{1}{4m}$	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							SMITH.	
41	0.628	0.81	60.13	1.03	0.469	0.4014		1 and 7
42	"	1.71	"	1.58	"	0.0368		1 and 7
43	"	3.31	"	2.30	"	0.0335	New gas pipe.....	1 and 7
44	"	4.87	"	2.86	"	0.0318		1 and 7
45	"	6.59	"	3.39	"	0.0306		1 and 7
46	"	8.35	"	3.88	"	0.0296		1 and 7
							SMITH.	
47	0.746	0.67	34.94	1.40	0.488	0.0367		8
48	"	2.06	"	2.63	"	0.0309	Glass pipe.....	8
49	"	3.66	"	3.67	"	0.0284		8
50	"	5.07	"	4.37	"	0.0277		8
							WESTON.	
51	0.969	* 121.10	781.80	6.03	.....	0.0221	Wrought-iron pipe, lined with tin.....	2 and 9
							WEISBACH.	
52	0.972	.....	.....	0.68	.....	0.0432		2 and 9
53	"	.....	.....	0.79	.....	0.0425		2 and 9
54	"	.....	.....	10.47	.....	0.0196	Zinc pipe.....	2 and 9
55	"	.....	.....	15.52	.....	0.0184		2 and 9
56	"	.....	.....	20.47	.....	0.0179		2 and 9
57	"	.....	.....	30.12	.....	0.0167		2 and 9
							HODSON.	
58	1.000	150.00	100.00	21.70	0.352	0.0160	....	2 and 9
							NEVILLE.	
59	1.020	11.13	9.29	14.58	0.352	0.0185	....	2 and 9
60	"	20.80	19.20	15.67	"	0.0182	....	2 and 9
							DARCY.	
61	1.047	* 0.07	328.09	0.12	.....	0.0838		25
62	"	0.08	"	0.13	.....	0.0836		25
63	"	0.08	"	0.13	.....	0.0799		25
64	"	0.11	"	0.19	.....	0.0515		25
65	"	0.18	"	0.28	.....	0.0388		25
66	"	0.50	"	0.43	.....	0.0461		25
67	"	1.60	"	0.81	.....	0.0413		25
68	"	3.33	"	1.21	.....	0.0391		25
69	"	6.36	"	1.71	.....	0.0371	New wrought-iron pipe..	25
70	"	6.26	"	2.19	.....	0.0367		25
71	"	14.27	"	2.61	.....	0.0358		25
72	"	20.72	"	3.15	.....	0.0357		25
73	"	22.88	"	4.05	.....	0.0343		25
74	"	34.68	"	4.20	.....	0.0336		25
75	"	58.49	"	5.52	.....	0.0329		25
76	"	83.99	"	6.56	.....	0.0335		25
77	"	101.55	"	7.17	.....	0.0339		25

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1.—*(Continued.)*

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	$\frac{1}{n}$	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							SMITH.	
78	1.054	0.47	60.17	0.96	0.563	0.0459		2 and 9
79	"	0.79	"	1.42	"	0.0345		2 and 9
80	"	1.66	"	2 15	"	0.0315		2 and 9
81	"	3.27	"	3.18	"	0.0282	New gas pipe, not tarred.	2 and 9
82	"	4.85	"	3.93	"	0.0269		2 and 9
83	"	6.59	"	4.67	"	0.0261		2 and 9
84	"	8.33	"	5.33	"	0.0253		2 and 9
		*					DARCY.	
85	1.055	0.07	328.09	0.10	.....	0.1285		
86	"	0.22	"	0.30	.....	0.0416		2 and 9
87	"	0.74	"	0.51	.....	0.0491		2 and 9
88	"	2.00	"	0.89	.....	0.0436		2 and 9
89	"	3.72	"	1 26	.....	0.0404		2 and 9
90	"	7.29	"	1.86	.....	0.0363	New sheet-iron pipe,	2 and 9
91	"	9.56	"	2.22	.....	0.0347	coated with bitumen.	2 and 9
92	"	14.90	"	2.80	.....	0.0328		2 and 9
93	"	38.87	"	4.81	.....	0.0289		2 and 9
94	"	59.01	"	6.10	.....	0.0274		2 and 9
95	"	80.12	"	7.23	.....	0.0265		2 and 9
96	"	100.77	"	8.22	.....	0.0258		2 and 9
97	1.063	0 07	164.05	0.21	.....	0.0552		2 and 9
98	"	0.49	"	0.62	.....	0.0450		2 and 9
99	"	1.34	"	1.09	.....	0.0391		2 and 9
100	"	3.72	"	1.96	.....	0.0337	New lead pipe.....	2 and 9
101	"	8.92	"	3.35	.....	0.0276		2 and 9
102	"	17.23	"	4.72	.....	0.0269		2 and 9
103	"	24.00	"	5.51	.....	0.0275		2 and 9
							BOSSUT.	
104	1.066	0.36	53.27	1.09	0.505	0.0299	Lead pipe.....	2 and 9
105	"	1.07	"	1.98	"	0.0267		2 and 9
							DUBUAT.	
106	1.066	0.01	65.45	0.14	0.503	0.0570		
107	"	0.40	65.45	0.82	"	0.0353		9
108	"	0.60	12.30	0.77	"	0.0378		2 and 9
109	"	0.37	12.30	0.93	"	0.1910		
110	"	0.37	12.30	0.95	"	0.1823		
111	"	0.53	65.45	1.18	"	0.0309		2 and 9
112	"	0.69	65.45	1.34	"	0.0315		2 and 9
113	"	0.80	65.45	1.45	"	0.0312		2 and 9
114	"	0.80	65.45	1.48	"	0.0300		2 and 9
115	"	1.09	65.45	1.78	"	0.0283	Tin pipe.....	2 and 9
116	"	1.22	65.45	1.86	"	0.0286		2 and 9
117	"	1.30	65.45	1.94	"	0.0280		2 and 9
118	"	2.10	65.45	2.55	"	0.0263		2 and 9
119	"	0.53	12.30	2.61	"	0.0256		2 and 9
120	"	1.60	10.39	5.18	"	0.0200		2 and 9
121	"	1.86	12.30	5.22	"	0.0209		2 and 9
122	"	2.37	10.39	6.33	"	0.0197		2 and 9
123	"	3.20	10.39	7.54	"	0.0181		2 and 9

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1.—*(Continued.)*

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							SMEATON.	
124	1.25	0.17	100.00	0.54	0.505	0.0375	.....	10
125	"	0.23	"	0.65	"	0.0350	.....	10
126	"	0.32	"	0.73	"	0.0387	.....	3 and 10
127	"	0.40	"	0.83	"	0.0374	.....	3 and 10
128	"	0.52	"	0.96	"	0.0363	.....	3 and 10
129	"	0.65	"	1.12	"	0.0332	.....	3 and 10
130	"	0.89	"	1.29	"	0.0343	.....	3 and 10
131	"	1.21	"	1.52	"	0.0336	.....	3 and 10
132	"	1.51	"	1.75	"	0.0315	.....	3 and 10
133	"	1.92	"	1.97	"	0.0316	.....	3 and 10
134	"	2.30	"	2.25	"	0.0289	.....	3 and 10
135	"	2.54	"	2.35	"	0.0293	.....	3 and 10
136	"	2.86	"	2.48	"	0.0296	.....	3 and 10
137	"	2.97	"	2.52	"	0.0298	.....	3 and 10
138	"	3.92	"	2.96	"	0.0285	.....	3 and 10
139	"	5.92	"	3.65	"	0.0282	.....	3 and 10
							SMITH.	
140	1.26	1.57	62.05	1.65	0.563	0.0599	Wood pipe.....	
141	"	3.31	"	2.47	"	0.0564		
142	"	4.94	"	3.01	"	0.0567		
143	"	6.70	"	3.52	"	0.0562		
144	"	8.52	"	3.99	"	0.0556		
							WEISBACH.	
145	1.27	.....	.....	1.18	.....	0.0343	Zinc pipe.....	3 and 10
							BOSSUT.	
146	1.42	1.07	191.78	1.12	0.505	0.0331	Tin pipe.....	3 and 11
147	"	1.07	159.82	1.25	"	0.0314		3 and 11
148	"	1.07	127.86	1.43	"	0.0296		3 and 11
149	"	2.13	191.78	1.62	"	0.0292		3 and 11
150	"	1.07	95.87	1.68	"	0.0281		3 and 11
151	"	2.13	159.82	1.87	"	0 0280		3 and 11
152	"	1.07	63.93	2.07	"	0.0267		3 and 11
153	"	2.13	127.86	2.13	"	0.0266		3 and 11
154	"	2.13	95.87	2.49	"	0.0254		3 and 11
155	"	1.07	31.96	2.94	"	0.0237		3 and 11
156	"	2.13	63.93	3.06	"	0.0243		3 and 11
157	"	2.13	31.96	4.31	"	0.0218		3 and 11
158	"	8.02	62.88	6.14	"	0.0228		3 and 11
159	"	15.16	125.76	6.15	"	0.0228		3 and 11
160	"	22.29	188.64	6.16	"	0.0228		3 and 11
							DARCY.	
161	1.43	* 0.08	328.09	0.15	.....	.....	Old cast-iron pipe, lined with deposit.	
162	"	0.23	"	0.26	.....	0.0809		26
163	"	0.60	"	0.42	.....	0.0813		26
164	"	2.20	"	0.81	.....	0.0792		26
165	"	5.00	"	1.22	.....	0.0793		26
166	"	10.63	"	1.76	.....	0.0805		26
167	"	13.63	"	2.02	.....	0.0782		26

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1.—*(Continued.)*

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							DARCY.	
168	1.43	* 0.23	328.09	0.37	.....	0.0396		27
169	"	0.59	"	0.62	.....	0.0363		27
170	"	2.14	"	1.27	.....	0.0309		27
171	"	4.73	"	1.97	.....	0.0285	The same, cleaned.....	27
172	"	9.90	"	2.93	.....	0.0271		27
173	"	13.01	"	3.39	.....	0.0265		27
174	"	15.26	"	3.69	.....	0.0262		27
							PROVIS.	
175	1.50	* 1.85	80.00	2 70	.....	0.0235		3 and 11
176	"	2.09	"	3.09	.....	0.0220		3 and 11
177	"	1.15	60.00	2.46	.....	0.0255		3 and 11
178	"	1.49	"	2.86	.....	0.0244		3 and 11
179	"	1.65	"	3.00	.....	0.0246		3 and 11
180	"	1.83	"	3.20	.....	0.0240		3 and 11
181	"	1.98	"	3.34	.....	0.0238		3 and 11
182	"	2.15	"	3.41	.....	0.0248		3 and 11
183	"	2.16	"	3 52	.....	0.0234		3 and 11
184	"	2.32	"	3.63	.....	0.0236		3 and 11
185	"	2.44	"	3.77	.....	0.0230		3 and 11
186	"	2.46	"	3.77	.....	0.0232		3 and 11
187	"	2.64	"	3.95	.....	0.0227		3 and 11
188	"	2.84	"	3.97	.....	0.0242		3 and 11
189	"	2.92	"	4.18	.....	0.0224		3 and 11
190	"	3.13	"	4.32	.....	0.0225		3 and 11
191	"	3.40	"	4.56	.....	0.0219		3 and 11
192	"	0.55	40.00	2.12	.....	0.0246		3 and 11
193	"	0.87	"	2.66	.....	0.0247	Lead pipe	3 and 11
194	"	0.90	"	2.68	.....	0.0252		3 and 11
195	"	1.16	"	3.13	.....	0.0238		3 and 11
196	"	1.18	"	3.16	.....	0.0238		3 and 11
197	"	1.20	"	3.18	.....	0.0239		3 and 11
198	"	1.45	"	3.52	.....	0.0235		3 and 11
199	"	1.46	"	3.58	.....	0.0229		3 and 11
200	"	1.49	"	3.58	.....	0.0234		3 and 11
201	"	1.50	"	3.64	.....	0.0228		3 and 11
202	"	1.66	"	3.90	.....	0.0219		3 and 11
203	"	1.76	"	3.93	.....	0 0229		3 and 11
204	"	1.77	"	3.98	.....	0.02.5		3 and 11
205	"	1.79	"	4.00	.....	0.0225		3 and 11
206	"	1.98	"	4.25	.....	0.0220		3 and 11
207	"	2.08	"	4.35	.....	0.0221		3 and 11
208	"	2.03	"	4.37	.....	0.0214		3 and 11
209	"	2.31	"	4 59	.....	0.0220		3 and 11
210	"	2.38	"	4.66	.....	0.0220		3 and 11
211	"	2.60	"	4.91	.....	0.0214		3 and 11
							DARCY.	
212	1.55	* 0.07	328.09	0.21	.....	0.0435		25
213	"	0.26	"	0.36	.....	0.0489		25
214	"	0.60	"	0.81	.....	0.0413		25
215	"	1.10	"	0.86	.....	0.0381	New wrought-iron pipe..	25
216	"	2.13	"	1.25	.....	0.0346		25
217	"	4.22	"	1.84	.....	0.0318		25
218	"	7.84	"	2.58	.....	0.0298		25

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1.—*(Continued.)*

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of influx.	ζ.	Name of the Experimenter, etc, and Nature of the Pipe.	No. of Diagram.
							DARCY.	
219	1·55	* 10.25	328.09	3.00	.....	0.0289		25
220	"	14.27	"	3.59	.....	0.0281		25
221	"	40.41	"	6.30	.....	0.0259	New wrought-iron pipe..	25
222	"	57.59	"	7.56	.....	0.0256		25
223	"	78.52	"	8.52	.....	0.0257		
224	1.61	0.13	164.05	0.39	....	0.0458		3 and 11
225	"	0.59	"	0.91	.....	0.0382		3 and 11
226	"	1.28	"	1.40	.....	0.0342		3 and 11
227	"	3.79	"	2.60	.....	0.0296	New lead pipe.....	3 and 11
228	"	9.19	"	4.32	.....	0.0260		3 and 11
229	"	18.17	"	6.32	.....	0.0240		3 and 11
230	"	24.97	"	7.56	.....	0 0230		3 and 11
231	1.96	0.14	147.19	0.50	.....	0.0399		4 and 12
232	"	0.51	"	1.02	.. ...	0.0345		4 and 12
233	"	1.14	"	1.59	.....	0.0320		4 and 12
234	"	3.41	"	2.93	.....	0.0283	New glass pipe	4 and 12
235	"	8.48	"	4.85	.....	0.0257		4 and 12
236	"	16.47	"	6.92	.....	0.0245		4 and 12
							Dr. ROBISON.	
237	2.00	12.75	3300.00	1.24	0.505	0.0277	....	4 and 12
							BOSSUT.	
238	2.14	1.07	191.78	1.45	0.505	0.0288		4 and 12
239	"	1.07	159.82	1.63	"	0.0273		4 and 12
240	"	1.07	127.86	1.84	"	0.0262		4 and 12
241	"	1.07	95.87	2.11	"	0.0258		4 and 12
242	"	2.13	191.78	2.20	"	0.0251		4 and 12
243	"	2.13	159.82	2.44	"	0.0241		4 and 12
244	"	1.07	63.93	2.59	"	0.0242	Tin pipe.....	4 and 12
245	"	2.13	127.86	2.74	"	0.0233		4 and 12
246	"	2.13	95.87	3.18	"	0.0225		4 and 12
247	"	1.07	31.96	3.58	"	0.0215		4 and 12
248	"	2.13	63.93	3.82	"	0 0211		4 and 12
249	"	2.13	31.96	5.23	"	0.0196		4 and 12
							LESLIE.	
250	2.50	0.46	1086.00	0.22	0.505	0.1212		
251	"	1.45	"	0.72	"	0.0347		4 and 13
252	"	2.78	"	1.09	"	0.0289		4 and 13
253	"	4.76	"	1.47	"	0.0271		4 and 13
254	"	7.01	"	1.73	"	0.0288		4 and 13
255	"	9.99	"	2.10	"	0.0278		4 and 13
256	"	1.01	540.00	0.58	"	0.0748		
257	"	1.78	"	0.79	"	0.0699		
258	"	2.50	"	1.07	"	0.0541	Lead pipe	4 and 13
259	"	5.50	"	2.21	"	0.0273		4 and 13
260	"	8.48	"	2.93	"	0.0239		4 and 13
261	"	9.99	"	3.35	"	0.0215		4 and 13
262	"	0.17	270.00	0.34	"	0.0700		
263	"	2.47	"	2.26	"	0.0229		4 and 13
264	"	5.44	"	3.26	"	0.0243		4 and 13
265	"	10 14	"	4.89	"	0.0199		4 and 13
266	"	0.19	100.00	0.86	"	0.0316		4 and 13

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1.—*(Continued.)*

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per Second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							LESLIE.	
267	2.50	4.36	100.00	4.89	0.505	0.0213	Lead pipe.....	4 and 13
268	"	6.61	"	6.01	"	0.0214		4 and 13
269	"	8.25	"	6.90	"	0.0201		4 and 13
							DARCY.	
270	3.15	* 0.21	328.09	0.40	.....	0.0693	Old cast-iron pipe, lined with deposit.....	26
271	"	0.82	"	0.81	.....	0.0641		26
272	"	2.38	"	1.44	.....	0.0589		26
273	"	5.28	"	2.19	.....	0.0568		26
274	"	10.17	"	3.01	.....	0.0580		26
275	"	14.88	"	3.69	.....	0.0564		26
276	"	0.28	"	0.63	.....	0.0358	The same cleaned.....	27
277	"	0.96	"	1.26	.....	0.0314		27
278	"	2.37	"	2.01	.....	0.0301		27
279	"	2.42	"	2.05	.....	0.0299		27
280	"	5.11	"	2.83	.....	0.0328		27
281	"	9.64	"	4.09	.....	0.0299		27
282	"	14.66	"	5.01	.....	0.0304		27
283	3.22	0.07	"	0.29	.....	0.0418	New cast-iron pipe.....	15
284	"	0.27	"	0.56	.....	0.0453		15
285	"	0.76	"	1.17	.....	0.0291		15
286	"	1.74	"	1.84	.....	0.0271		15
287	"	3.35	"	2.59	.....	0.0262		15
288	"	7.40	"	3.89	.....	0.0258		15
289	"	10.53	"	4.65	.....	0.0256		15
290	"	13.26	"	5.15	.....	0.0263		15
291	"	31.32	"	8.05	.....	0.0255		15
292	"	32.49	"	8.16	.....	0.0257		15
293	"	39.30	"	8.92	.....	0.0260		15
294	"	55.14	"	10.62	.....	0.0258		15
295	"	56.01	"	10.71	.....	0.0257		15
296	3.25	0.09	"	0.33	.....	0.0438	New sheet-iron pipe, coated with bitumen.....	5 and 14
297	"	0.22	"	0.58	.....	0.0345		5 and 14
298	"	0.67	"	1.17	.....	0.0258		5 and 14
299	"	2.06	"	2.18	.....	0.0231		5 and 14
300	"	4.00	"	3.12	.....	0.0219		5 and 14
301	"	7.50	"	4.44	.....	0.0202		5 and 14
302	"	10.19	"	5.29	.....	0.0194		5 and 14
303	"	13.35	"	6.15	.....	0.0188		5 and 14
304	"	23.52	"	8.44	.....	0.0176		5 and 14
305	"	34.96	"	10.53	.....	0.0167		5 and 14
306	"	45.54	"	12.03	.....	0.0167		5 and 14
307	"	51.20	"	12.79	.....	0.0167		5 and 14
							JARDINE.	
308	4.50	51.00	14930.00	1.71	0.505	0.0282	Lead pipe.....	
							COUPLET.	
309	5.38	0.50	7481.66	0.18	0.505	0.0593	Stoneware and lead pipe.	16
310	"	1.01	"	0.28	"	0.0489		16
311	"	1.49	"	0.37	"	0.0422		16
312	"	1.87	"	0.43	"	0.0391		16
313	"	2.13	"	0.46	"	0.0379		16
314	"	2.22	"	0.47	"	0.0378		16

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1—(*Continued.*)

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	$\frac{1}{n}$	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							DARCY.	
315	5.39	0.08	328.09	0.49	.....	0.0292		16
316	"	0.29	"	0.98	.....	0.0263		16
317	"	0.69	"	1.60	.....	0.0236		16
318	"	1.56	"	2.50	.....	0.0220		16
319	"	4.13	"	4.19	.....	0.0208	New cast-iron pipe.....	16
320	"	7.30	"	5.62	.....	0.0204		16
321	"	10.89	"	6.88	.....	0.0203		16
322	"	12.81	"	7.48	.....	0.0202		16
323	"	32.32	"	11.94	.....	0.0200		16
324	"	54.98	"	15.39	.....	0.0205		16
							WESTON.	
325	6.00	* 21.19	1170.90	4.70	.....	0.0264		16
326	"	47.64	"	7.25	.....	0.0249	Cast-iron pipe, coated with coal-tar, in service 4 years.....	16
327	"	65.46	"	8.49	.....	0.0249		16
328	"	73.16	"	9.26	.....	0.0234		16
							DARCY.	
329	7.40	* 0.09	328.09	0.65	.....	0.0251		17
330	"	0.57	"	1.64	.....	0.0259		17
331	"	1.21	"	2.50	.....	0.0233		17
332	"	2.64	"	3.72	.....	0.0231		17
333	"	4.40	"	4.90	.....	0.0221	New cast-iron pipe	17
334	"	7.38	"	6.37	.....	0.0220		17
335	"	12.50	"	8.25	.....	0.0221		17
336	"	36.02	"	14.23	.....	0.0215		17
337	"	47.87	"	16.23	.....	0.0219		17
338	7.72	0.07	"	0.59	.....	0.0237		23
339	"	0.16	"	0.91	.....	0.0239		23
340	"	0.42	"	1.53	.....	0.0228		23
341	"	1.08	"	2.56	.....	0.0209		23
342	"	1.90	"	3.53	.....	0.0193		23
343	"	3.90	"	5.44	.....	0.0167	New sheet-iron pipe, coated with bitumen.	23
344	"	3.94	"	5.51	.....	0.0164		23
345	"	6.89	"	7.41	.....	0.0158		23
346	"	9.74	"	9.00	.....	0.0152		23
347	"	11.94	"	10.01	.....	0.0150		23
348	"	39.88	"	19.72	.....	0.0129		23
349	9.63	0.31	"	1.00	.....	0.0489		26
350	"	0.66	"	1.46	.....	0.0487		26
351	"	1.55	"	2.29	.....	0.0465		26
352	"	3.77	"	3.58	.....	0.0463		26
353	"	7.51	"	5.01	.....	0.0471	Old cast-iron pipe, lined with deposit	26
354	"	10.50	"	5.94	.....	0.0468		26
355	"	18.47	"	6.72	.....	0.0470		26
356	"	45.87	"	12.42	.....	0.0468		26
357	"	0.17	"	0.91	.....	0.0316		27
358	"	0.54	"	1.76	.....	0.0275		27
359	"	1.63	"	3.11	.....	0.0266		27
360	"	3.79	"	4.66	.....	0.0274		27
361	"	6.68	"	6.25	.....	0.0269	The same cleaned.....	27
362	"	8.97	"	7.24	.....	0.0270		27
363	"	12.24	"	8.44	.....	0.0271		27
364	"	37.22	"	14.75	.....	0.0269		27

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1—(*Continued.*)

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							SMITH.	
365	10.93	6.56	730.60	4.76	.....	0.0219	Riveted sheet-iron pipe, coated with coal-tar and asphaltum.....	18
366	"	10.20	721.30	6.12	.....	0.0209		18
367	"	12.85	713.90	6.95	.....	0.0206		18
368	"	19.01	697.00	8.69	.....	0.0198		18
369	"	24.22	684.80	10.05	.....	0.0192		18
							DARCY.	
370	11.22	* 0.23	328.09	1.30	.....	0.0251	New sheet-iron pipe, coated with bitumen.	24
371	"	0.84	"	2.78	.....	0.0198		24
372	"	1.42	"	3.87	.....	0.0174		24
373	"	2.25	"	4.90	.....	0.0172		24
374	"	3.90	"	6.67	.....	0.0161		24
375	"	6.71	"	8.85	.....	0.0157		24
376	"	9.21	"	10.52	.....	0.0153		24
377	11.69	0.09	"	0.80	.....	0.0275	Old cast-iron pipe, thoroughly cleaned.....	18
378	"	0.39	"	1.76	.....	0.0240		18
379	"	0.88	"	2.71	.....	0.0229		18
380	"	1.76	"	3.79	.....	0.0235		18
381	"	3.63	"	5.42	.....	0.0238		18
382	"	7.56	"	7.84	.....	0.0236		18
383	"	10.52	"	9.18	.....	0.0239		18
384	"	13.47	"	10.37	.....	0.0240		18
							SIMPSON.	
385	12.00	4.00	5200.00	1.45	0.505	0.0232	Cast-iron pipe, in service less than seven years...	18
386	"	19.00	"	2.91	"	0.0276		18
387	"	16.75	"	2.91	"	0.0243		18
388	"	40.00	"	4.35	"	0.0259		18
389	"	38.00	"	4.35	"	0.0246		18
390	"	5.50	8140.00	1.42	"	0.0213	Cast-iron pipe, no record of service	18
391	"	12.00	"	1.89	"	0.0264		18
392	"	18.00	"	2.21	"	0.0291		18
393	"	24.75	"	2.53	"	0.0305		18
394	"	27.00	"	2.64	"	0.0304		18
395	"	34.50	6600.00	3.57	"	0.0263	Cast-iron pipe, in service less than 4 years.....	18
396	"	46.00	"	4.11	"	0.0277		18
							BONN WATER WORKS.	
397	12.05	* 21.30	17684.00	1.55	.....	0.0322	New cast-iron pipe, coated with asphaltum	
398	"	21.30	"	1.57	.....	0.0317		
399	"	34.40	"	2.10	.....	0.0284		
400	"	32.80	"	2.12	.....	0.0267		
401	"	47.60	"	2.60	.....	0.0257		
402	"	45.90	"	2.62	.....	0.0244		
403	"	64.00	"	3.10	.....	0.0244		
404	"	60.70	"	3.13	.....	0.0227		
							SMITH.	
405	12.67	5.13	718.40	4.61	.....	0.0213	Riveted sheet-iron pipe, coated with coal-tar and asphaltum.....	18
406	"	10.89	709.20	6.98	.....	0.0199		18
407	"	16.69	699.60	8.68	.....	0.0200		18
408	"	24.51	684.90	10.76	.....	0.0194		18

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1—(*Continued.*)

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per Second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							SMITH.	
409	14.76	3.92	719.90	4.40	.....	0.0205	Riveted sheet-iron pipe, coated with coal-tar and asphaltum.....	19
410	"	8.55	712.40	6.86	.....	0.0184		19
411	"	9.55	710.70	7.33	.....	0.0180		19
412	"	12.72	705.00	8.52	.....	0.0179		19
413	"	18.93	695.60	10.75	.....	0.0169		19
414	"	24.39	684.40	12.30	.....	0.0168		19
							EDINBURGH WATER CO.	
415	16.00	230.00	25765.00	5.25	0.505	0.0277	Cast-iron pipe, in service 8 or 9 years.....	19
416	"	420.00	29580.00	6.82	"	0.0261		19
417	"	184.00	3815.00	14.51	"	0.0191		19
							LAMPE.	
418	16.48	15·09 *	25414.00	1.58	.....	0.0210	New cast-iron pipe, coated with varnish.....	19
419	"	43.65	31719.00	2.48	.....	0.0197		19
420	"	51 69	31719.00	2.71	.....	0.0196		19
421	"	52.39	26862.00	3.09	.....	0.0180		19
							SMITH.	
422	16.99	303.60	4438.70	20.13	0.182	0.0150	Riveted sheet-iron pipe, coated with coal-tar, etc.	19
							SIMPSON.	
423	19.00	24.00	22444.00	2.06	0.505	0.0257	Cast-iron pipe, in service less than thirteen years.	20
424	"	27.50	"	2.26	"	0.0244		20
425	"	34.00	"	2.52	"	0.0242		20
426	"	41.00	"	2.73	"	0.0248		20
427	"	43.50	"	2.80	"	0.0251		20
							DARCY.	
428	19.69	0.15 *	328.09	1.38	.....	0.0252	New cast-iron pipe.....	20
429	"	0.15	"	1.47	.....	0.0222		20
430	"	0·20	"	1.55	.....	0.0263		20
431	"	0.39	"	2.60	.....	0.0189		20
432	"	0.41	"	2.60	.....	0.0195		20
433	"	0.69	"	3.42	.....	0.0190		20
434	"	0.76	"	3.64	.....	0.0183		20
435	"	0.85	"	3.66	.....	0.0205		20
436	"	0.82	"	3.69	.....	0.0194		20
							FANNING.	
437	20.00	.....	8171.00	0.95	.....	0.0271	Wrought-iron cement lined pipe.....	20
438	"	.....	"	1.49	.....	0.0214		20
439	"	.....	"	1.93	.....	0.0210		20
440	"	.....	"	2.33	.....	0.0205		20
441	"	.....	"	2·60	.....	0.0213		20
442	"	.....	"	2.87	.....	0.0206		20
443	"	.....	"	3.27	.....	0.0200		20
444	"	.....	"	3.44	.....	0.0207		20
445	"	.....	"	3.74	.....	0.0209		20
446	"	.....	"	3.92	.....	0.0210		20

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1—(*Continued.*)

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per Second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							FANNING.	
447	20.00	.....	8171.00	4.00	.....	0.0210	Wrought-iron cement lined pipe.....	20
448	"	.....	"	4.04	.....	0.0211		20
							BRUSH.	
449	20.00	0.729	1000.00	2.00	.....	0.0197	Cast-iron pipe, coated with tar, in service five years.....	20
450	"	0.876	"	2.24	.....	0.0188		20
451	"	1.029	"	2.36	.....	0.0198		20
452	"	1.186	"	2.52	.....	0.0200		20
453	"	1.337	"	2.68	.....	0.0199		20
454	"	1.490	"	2.76	.....	0.0210		20
455	"	1.645	"	2.92	.....	0.0206		20
456	"	1.797	"	3.00	.....	0·0214		20
							BAILEY.	
457	20.00	28.13	29715.00	1.44	0.505	0.0491	Cast-iron pipe, tuberculated, in service two years.....	26
							DARRACH.	
458	20.00	16.75	4320.00	2.71	.....	0.0568	Cast-iron pipe, in service eleven years.....	26
459	"	19.64	"	3.01	.....	0.0539		26
460	"	22.52	"	3.31	.....	0.0508		26
461	"	25.42	"	3.61	.....	0.0482		26
462	"	28.31	"	3.91	.....	0.0459		26
463	"	31.19	"	4.21	.....	0.0437		26
464	"	34.08	"	4.51	.....	0.0413		26
465	"	36.96	"	4.81	.....	0.0394		26
466	"	39.84	"	5.11	.....	0.0376		26
							SIMPSON.	
467	30.00	25.00	54120.00	1.77	0.505	0.0237	Cast-iron pipe, in service two or three years.....	21
							DARRACH.	
468	30.00	1.55	4000.00	1.60	.....	0.0242	New cast-iron pipe. The head due to friction, does not include the resistance of two check valves which were on this pipe.....	21
469	"	1.83	"	1.74	.....	0 0242		21
470	"	2.11	"	1.87	.....	0.0241		21
471	"	2.40	"	2.00	.....	0.0240		21
472	"	2.68	"	2.14	.....	0.0236		21
473	"	6.27	20200.00	1.47	.....	0.0230	Cast-iron pipe, in service two years. The head due to friction, does not include the resistance of four check valves which were on this pipe.....	21
474	"	7.57	"	1.62	.....	0.0228		21
475	"	8.87	"	1.76	.....	0.0224		21
476	"	10.17	"	1.91	.....	0.0221		21
477	"	11.44	"	2.06	.....	0.0214		21
478	"	12.77	"	2.20	.....	0.0209		21
479	"	14.07	"	2.35	.....	0.0201		21
480	"	15.37	"	2.50	.....	0.0185		21
481	"	16.67	"	2.64	.....	0.0189		21
482	"	17.96	"	2.79	.....	0.0181		21
483	"	19.26	"	2.94	.....	0.0176		21
484	"	20.56	"	3.08	.....	0.0172		21
485	"	21.86	"	3.23	.....	0.0166		21

* The heads of this authority are those due to friction only.

Digitized by Google

TABLE No. 1—(*Continued.*)

No.	Diameter in Inches.	Head in Feet.	Length in Feet.	Velocity in Feet per Second.	Co-efficient of Influx.	ζ.	Name of the Experimenter, etc., and Nature of the Pipe.	No. of Diagram.
							DARRACH.	
486	30.00	* 4.04	4400.00	1.07	.....	0.1283		
487	"	4.62	"	1.21	.....	0.1160		
488	"	5.20	"	1.34	.....	0.1053		
489	"	5.78	"	1.48	.....	0.0966		
490	"	6.35	"	1.61	.....	0.0892	Cast-iron pipe, in service nine years.....	
491	"	6.93	"	1.75	.....	0 0828		
492	"	7.51	"	1.88	.....	0.0772		
493	"	8.09	"	2.02	.....	0.0724		
494	"	8.66	"	2.15	.....	0.0681		
495	"	9.24	"	2.29	.....	0.0644		
							GREENE.	
496	36.00	20.22	11217.00	3.00	0.505	0.0383	Cast-iron pipe, heavily tuberculated	28
							DARRACH.	
497	36.00	* 3.45	3700.00	1.58	.....	0.0716		28
498	"	4.03	"	1.74	.....	0.0687		28
499	"	4.61	"	1.89	.....	0.0669	Cast-iron pipe, in service	28
500	"	5.19	"	2.05	.....	0.0640	seven years	28
501	"	5.77	"	2.21	.....	0.0614		28
502	"	6 35	"	2.37	.....	0.0586		28
503	"	13.00	12400.00	1.00	.....	0.2013		28
504	"	13.28	"	1.11	.....	0.1668		28
505	"	13.57	"	1.22	.....	0.1402		28
506	"	13.86	"	1.33	.....	0.1208		28
507	"	14.15	"	1.44	.....	0.1050	Cast-iron pipe, in service	28
508	"	14.44	"	1.56	.....	0.0926	seven years.....	28
509	"	14.73	"	1.67	.....	0.0822		28
510	"	15.01	"	1.78	.....	0.0739		28
511	"	15.30	"	1.89	.....	0.0667		28
512	"	15.59	"	2.00	.....	0.0604		28
							STEARNS.	
513	48.00	* 0.56	1747.20	2.62	.....	0.0120	Cast-iron pipe, coated	21
514	"	1.24	"	3.74	.....	0.0131	with coal-tar, in service	21
515	"	2.13	"	4.97	.....	0.0128	three years	21
516	"	3.23	"	6.20	.....	0.0124		
							GALE.	
517	48.00	* 5.00	5280.00	3.46	.....	0.0204	Cast-iron pipe, coated with coal-tar, in service eight years.....	21
							CLARKE.	
518	90.00	* 3.68	7166.00	3.77	.....	0.0174		22
519	"	3.98	"	3.80	.....	0.0186	Brick tunnel.....	22
520	"	4.17	"	3.93	.....	0.0182		22

* The heads of this authority are those due to friction only.

Digitized by Google

DESCRIPTION OF THE EXPERIMENTS THE RESULTS OF WHICH ARE CONTAINED IN TABLE NO. 1.

All of the experiments that were made under the direction of one authority are described together, and each description is preceded by the title of the work or publication, from which the information upon which it is based, was obtained.

Experiments of Bossut.

"Traité théorique et expérimental d'Hydro-dynamique," by the Abbé Bossut, Paris, 1786.

The twenty-nine experiments were made under the direction of the Abbé Bossut, an eminent French geometer, Professor of Mathematics in the School of Engineers at Mezières, and Member of the Academy of Sciences, with lead pipes, 1.07, 1.42 and 2.14 inches in diameter, which in all cases discharged into the open air. Their tabular numbers are, 1.07 inch, 104 to 105; 1.42 inch, 146 to 160 inclusive; 2.14 inch, 238 to 249 inclusive.

Those numbered in Table No. 1, 104 and 105, 146 to 157, and 238 to 249 inclusive, were included in the fifty-one experiments that were used by Prony in determining his formula, and were made with straight pipes. These pipes were supplied with water from a closed metal box 1 foot square, that was connected by a pipe about 8 inches in diameter with a supply reservoir, in which the water was maintained at a constant elevation.

The other experiments that were made by M. Bossut, which are numbered in Table No. 1, 158 to 160 inclusive, were made with a pipe 1.42 inches in diameter that was laid on an incline of 6° 30′. The upper end of the pipe was connected with a supply reservoir, in which the water was kept at a constant head of about 0.89 feet above the center of the inlet end of the pipe; this head being equivalent to the head due to the velocity, and the loss arising from contraction at the inlet of the pipe. Three separate lengths of pipe were used in making these experiments, the inclination being the same in each instance.

Experiments of Bailey.

"The Brooklyn Water Works and Sewers," by James P. Kirkwood. New York, 1867.

This experiment was made by George H. Bailey, C. E., with a compound cast-iron main 29 715 feet long, belonging to the Jersey City Water Works. Its tabular number is 457.

"This main was 20 inches in diameter for a length of 29 587 feet, and 24 inches in diameter for a length of 128 feet. It connected the

Digitized by Google

receiving with the distributing reservoir, and at the time the experiment was made is said to have been from one to two years old. The head was ascertained by leveling between the reservoir surfaces, and the flow was measured by the lowering of the surface of the water in the feeding reservoir.

Six experiments were really made, but their average only, as published by Mr. Kirkwood, is given in Table No. 1, and the diameter was used by the writer as 20 inches for the entire length of the pipe, in computing the co-efficient of friction ζ.

Experiments of Brush.

"*Transactions of the American Society of Civil Engineers*," *Vol. XIX*, 1888.

The eight experiments were made under the direction of Charles B. Brush, M. Am. Soc. C. E., with a cast-iron pumping main, 20 inches in diameter and 75 000 feet long, belonging to the Hackensack Water Company. Their tabular numbers are 449 to 456 inclusive.

This main was laid along by the side of railroads, and in highways, with a large number of summits, angles and curves. Among the horizontal bends there were four right angles and ten quadrants of about 30 feet radius. It had been in service about five years when the experiments were made, and there was not any oxidization upon the interior surface. The pipes, when new, were coated in the usual way with tar. A pressure gauge was used in determining the head. The flow of water was measured by the displacement of the pump plungers of a Worthington pumping engine, 5 per cent. having been allowed for slip. The average delivery of the main was about 4 000 000 gallons per day, but 200 000 of this amount was supplied from the main to consumers, before the balance reached the reservoir, 100 000 gallons being taken at a point 13 500 feet from the pumping station, and the remaining 100 000 at a point 70 000 feet from the pumping station. The "effective" head ranged from 55 to 135 feet according to the quantity of water flowing in the main. The head given in Table No. 1 is the frictional head per 1 000 feet as given by Mr. Brush.

Bonn Water Works Experiments.

"*Hydraulics*," *by Hamilton Smith, Jr., M. Am. Soc. C. E. New York*, 1886.

The experiments were made with a new cast-iron force main, coated with asphaltum, 12.48 inches in diameter and 17 684 feet long, belonging to the Bonn Water Works. Their tabular numbers are 397 to 404 inclusive.

Digitized by Google

The pipe was "supplied by pumps at the lower end;" it had "no obstructions of consequence, such as valves or sharp bends." The discharge "was determined by absolute measurement in a reservoir, and also by capacity of the plungers of the pumps; the two measurements agreed closely. Owing to the disturbed condition of the surface of the water in the reservoir" the quantity "was probably not very accurately determined. The head was determined by manometers at the pumps (probably Bourdon gauges attached to the air-chambers of the two pumps). There was a small amount of air in the pipe, which may have affected accuracy of results."

Experiments of Couplet.

"Recherches sur le Mouvement des Eaux," by M. Couplet, Memoirs of the Academy of Sciences, 1732.

Of the fifteen experiments that were made at Versailles under the direction of M. Couplet, the writer has selected only six of the seven that were included in the fifty-one experiments that were used by Prony in determining his formula. The six experiments were made with a pipe 5.33 inches in diameter and 7481.7 feet long; for a distance of about 320 feet the pipe was of stone-ware, and for the remaining distance of lead. Their tabular numbers are 309 to 314 inclusive.

There were several slight turns, and one quite abrupt one, in the pipe. The difference between the elevation of the supply tank and the vertical outlet of the pipe was ascertained by connecting a temporary vertical pipe to the outlet, and allowing the quiescent water to take its hydrostatic level. The flow was determined by dividing the discharging jet of water into two parts, and measuring each one separately in a vessel having a capacity of about 0.63 cubic feet.

Experiments of Clarke.

"Main Drainage Works of the City of Boston," by Eliot C. Clarke, M. Am. Soc. C. E. Third Edition, 1888.

The experiments were made under the direction of Mr. Clarke, with the Main Drainage Tunnel belonging to the City of Boston, which extends under the harbor from the mainland at Dorchester to Squantum Neck. Mr. Clarke mentions seven experiments, but the writer has selected only the three during which the flow seems to have been determined with accuracy. Their tabular numbers are 518 to 520 inclusive.

The tunnel is circular in form, 90 inches in diameter, 7 166 feet long, and lined with hard brick. It is flushed once in about two weeks with clean salt water, by utilizing the entire pumping capacity of the engines. At the time the experiments were made the tunnel was flowing full, and the volume consisted of about three-fourths salt water and one-fourth sewage.

Digitized by Google

The flow through the tunnel is generated by the difference in level of the elevation of the water at its two extremities. This difference, which is the frictional resistance, was carefully ascertained by knife-edged sliding gauges. The head was taken a short distance below the entrance of the tunnel during the three experiments, in order to avoid a correction for entrance. The flow was measured during Experiment No. 520, by the displacement of the pump plungers of the pumping engines, allowance having been made for a slip, which was ascertained by trial previous to the experiment in the reservoir, in which the flow during Experiments Nos. 518 and 519 were measured.

The section of the tunnel at its farther end increases gradually, similar to a diverging tube, which somewhat reduces the resistance, while on the other hand, the tunnel has a quarter turn of 9.75 feet radius and one angle of 23° 15′, which slightly increases the resistance.

Experiments of Darcy.

"*Recherches experimentales relatives au mouvement de l'Eau dans les Tuyaux*," *by Henry Darcy, Paris*, 1857.

The experiments were made under the direction of Henry Darcy, an eminent French Civil Engineer, while he had charge of the water service of the City of Paris, during the years 1849, 1850 and 1851. They will be described at a considerable length, on account of the completeness of the apparatus that was used, and the thorough and elaborate manner in which they were conducted.

The synoptic table on page 27 gives the nomenclature, etc., of the different kinds of pipes that were used in making the 203 experiments.

At Chaillot, where the experiments were made, the natural facilities were very favorable. In a northerly direction from the building containing the pumping machinery, which was located on the north bank of the Seine, there was a circular sheet-iron tank, having a capacity of 105 677 gallons, elevated about 134.50 feet above the ground where the experimental plant was set up. In the same direction there were also four large reservoirs of different elevations, which could be connected with each other or isolated at will, each having a capacity of 873 589 gallons; their respective elevations were 77.10 feet, 81.70 feet, 83.99 feet, and 86.95 feet.

The circular tank was supplied by the pumping machinery through a conduit 9.84 inches in diameter, and the four reservoirs were supplied from the same source through a conduit 25.59 inches in diameter.

In the rear of the building containing the pumping machinery there was a cistern, suitable for measuring large quantities of water, which was ordinarily used to receive condensation water.

Digitized by Google

Number of experiments made.	Diameter in Inches.	Nature of the Pipes.	Total Length in Feet.	Length of each Pipe in Feet.	Nature of the Joints.	As numbered in Table No. 1.
13	0.480	New wrought-iron pipe.....	374.61	7.05	Screw..	15 to 27
17	1.047	" "	372.23	8.10	"	61 to 77
12	1.550	" "	371.92	12 63	"	212 to 223
7	0.551	New lead pipe.....	172.05	38.55	Solder.	34 to 40
7	1.063	" "	172.38	21.65	"	97 to 103
7	1.610	" "	172 41	17.06	"	224 to 230
12	1.055	New sheet-iron pipe, coated with bitumen.	371.86	9.15	Screw..	85 to 96
12	3.25	" "	365.07	9 51	"	296 to 307
11	7.72	" "	365 32	9.51	"	338 to 348
7	11.22	" "	365.47	9.51	"	370 to 376
6	1.96	New glass pipe.....	184.56	3.81	Flange.	231 to 236
13	3.22	New cast-iron pipe	366.10	8.20	Socket.	283 to 295
10	5.39	" "	365 74	8.20	"	315 to 324
9	7.40	" "	365.40	8.20	"	329 to 337
9	19.69	" "	365.34	8.20	"	428 to 436
14	1.43	Old cast-iron pipe, lined with deposit, and the same cleaned ..	374.94	4.27	Flange.	161 to 174
13	3.15	" "	366.31	8.20	Socket.	270 to 282
16	9.63	" "	365.35	8.20	Flange.	349 to 364
8	11.69	Old cast-iron pipe, thoroughly cleaned.	365.28	8.20	Socket.	377 to 384

The water used in making the experiments was taken from the 25.59 and 9.84-inch conduits, at a point about 325 feet north of the pump building. A vertical pipe of the same diameter as the conduit, and of suitable length, was connected to the 25.59-inch conduit; there was connected at a right angle, to the upper part of this vertical pipe, a conduit 11.81 inches in diameter; upon the 11.81-inch conduit there was a gate about 3 feet from the vertical pipe; two lead pipes 0.55 and 1.61 inches in diameter were arranged as connections between the vertical pipe and the 11.81-inch conduit, beyond the gate. The 9.84-inch conduit was also connected with the 11.81-inch conduit, beyond the gate, by a lead pipe 1.06 inches in diameter. Each of these small pipes was supplied with a stop-cock.

When the four large reservoirs were used, and small volumes required, the water was taken through the 0.55-inch pipe, its stop-cock having been opened more or less, dependent upon the requirements of the experiments; if the flow was not sufficient through this pipe it was shut off and the 1.61-inch pipe brought into service, which in due course was abandoned for the 11.81-inch conduit.

In this manner all the heads were utilized that could be furnished by the reservoir having the lowest elevation, which was maintained at a constant level by means of the water that was contained in the three others more elevated.

Digitized by Google

To obtain greater heads, communication was established with the sheet-iron circular tank, by opening the stop-cock on the 1.06-inch pipe, through which the water passed from the 9.84-inch conduit, which was supplied by this tank.

About 48 feet beyond the gate the 11.81-inch conduit was connected with a horizontal cast-iron cylinder 11.50 feet long, and 3.28 feet in diameter.

The horizontal cylinder formed an intermediate reservoir, and was intended to deaden the velocity of the water before it entered the experimental conduits. In the interior there was a vertical sheet-iron diaphragm perforated with holes, through which the water passed after its entrance from the 11.81-inch conduit. One end of the cylinder consisted of a semi-circular cap, and the other of a vertical plate to which the experimental conduits were connected.

The conduits used in making the experiments were laid upon blocks of masonry, on an incline inverse to the flow, in order to more easily free them from air.

The outlet end of each conduit discharged under water and was connected to the lower part of one of two vertical cast-iron cylinders. The bottoms of the cylinders were closed and the tops open. Each conduit had a gate or stop-cock located upon it at a short distance from where it entered its respective cylinder. These outlet cylinders were not of the same size, the larger was 5.30 feet in diameter and 10.93 feet high, and the smaller was 1.05 feet in diameter and 6.56 feet high. Only one of them was used at one time, the size depending upon the diameter of the conduit being experimented upon, and the volume of water flowing. A double communication was established between the large cylinder and the gauging basins; one of these communications was composed of a pipe 4 inches in diameter, which served to carry the water to a small gauging basin when small volumes were used; the other consisted of a notch 2.62 feet wide and 1 foot deep, at the top of the cylinder, into which was fitted a wooden channel which carried the water to the large gauging basins in the rear of the pump building; the 4-inch pipe was connected at a lower elevation than the channel, and when the capacity of the small gauging basin was insufficient, the inlet was stopped with a wooden plug, and the water rose in the cylinder and escaped by the channel. The water which entered the small cylinder was conducted to one of three small gauging basins, dependent upon the volume of water flowing, by means of a pipe 3.19 inches in diameter; this pipe had a movable elbow by which the water could be directed into either of the three, at will.

Seven gauging basins could be employed for measuring the water discharged by the experimental conduits. In order to be described more clearly, these basins will be numbered 1, 2, 3, 4, 5, 6 and 7. Basins Nos. 1, 2, 3 and 4 were formed by masonry division walls, which were

Digitized by Google

located in the large rectangular cistern in the rear of the pump building; one wall divided the cistern into two equal parts, and another subdivided the one of these parts nearest to the pump building; by this arrangement there were available three basins, Nos. 1 and 2 each having an area about one-fourth of the total area of the cistern, and No. 3 having an area about one-half of the total area of the cistern; the wooden channel emptied into No. 1, which was located nearest to the vertical outlet cylinders; No. 4 was composed of the upper part of the cistern, from the level of the summit of the dividing walls to a point about 8.20 feet above these walls.

Two float gauges, located in Nos. 1 and 3, were used to measure the height of the water. Basins Nos. 1, 2 and 3 drained into the Seine, and could be connected or isolated at will; measurements were not made in Nos. 1, 2 and 3 until the water had covered their inverts, and not in No. 4, which was used only in making experiments with conduits of the largest diameter, until the water had reached a level above the division walls. Great care was taken in ascertaining the capacity of these basins, and in determining the amount of water that was lost by filtration and leakage from them; the amount lost in this way, however, was generally very insignificant.

The small gauging basins Nos. 5, 6 and 7, which were located quite near the outlet cylinders, were vertical cylinders with open tops; Nos. 5 and 6 were made of cast-iron, and No. 7 of lead; they all had stop-cocks or plugs, conveniently arranged for letting out the water after each experiment. On the outside of each there was adjusted a glass tube, which was connected with the interior, for measuring the elevation of the water, which was accomplished by placing a scale about 6.50 feet long against the tubes. Their respective diameters and heights were as follows:

	Diameter.	Height.
No. 5	3.15 feet	7.45 feet.
No. 6	1.08 "	8.20 "
No. 7	0.33 "	8.20 "

Five water and five mercurial manometers were used to determine the heads, and each set, to facilitate description, will be numbered 1, 2, 3, 4 and 5. (In reality, however, there were but four different manometric tubes, etc., as Nos. 3 and 4 consisted of one tube, etc., which was connected to the experimental conduits at two different points.) They were all set up against one post, which was located about half way between the horizontal inlet and vertical outlet cylinders, in order to be able to compare their indicated results without a possibility of error. The manometers were connected at different points to the experimental conduits, and to the horizontal cylinder, by lead pipes 0.55 inches in diameter, which were laid upon planks placed upon the masonry blocks that support the experimental conduits. A ladder was erected by the side of the post for convenience in observing the manometers.

Digitized by Google

Small closed reservoirs of a capacity of about 0.6 of a cubic foot were established upon the 0.55 inch manometer pipes near their junctions with the experimental conduits, with the exception of the pipe connecting manometer No. 5, the reservoir of which was located near the foot of the manometer post; in the upper part of these reservoirs, as well as at the highest points of the 0.55-inch pipes, the experimental conduits, the manometers, and the horizontal cylinder, air vents were established.

Water manometers were employed for all heads that did not exceed 19.69 feet, and mercurial manometers for heads above 19.69 feet.

The water manometers were adjusted upon a vertical plank graduated to millimeters, which was fastened to the front of the post facing the experimental conduits.

Each water manometer consisted of a vertical glass tube about 18 feet in length, formed by joining together in a copper fitting, with gum-lac, two glass tubes about 9 feet in length. They were all operated in the same manner, and had stop cocks at their junctions with the 0.55 inch connecting pipes.

The mercurial manometers were adjusted upon two vertical planks graduated to millimeters, which were erected behind the post; they were connected to the same 0.55 inch lead pipes that served as connections between the water manometers and the experimental conduits; and stop-cocks were so arranged that either the water or the mercurial manometers could be used at will.

Each mercurial manometer consisted of the following combination. A straight lead pipe rose to a height of about 6.30 feet above the bottom of the graduated plank; one end of a copper tube, about 4 inches long, was connected at a right angle with the upper end of the lead pipe by an elbow; a glass tube of the same length as the lead pipe was connected at a right angle with the other end of the copper tube by an elbow, and ran down in a direction parallel to the lead pipe to the bottom of the plank; one end of an iron tube, about 4 inches long, was connected at a right angle with the lower end of the glass tube; a second glass tube about 9 feet in length was connected at a right angle with the other end of the iron tube, by an elbow, and ran up parallel to the other glass tube and the lead pipe to the top of the plank. The iron tube had connected to its lower side a rubber pocket, containing mercury, which was introduced by pressure into the two glass tubes; this pocket, which had a stop-cock upon its connection with the iron tube, also served to receive the mercury when it was necessary to discharge the manometer. The copper tube had a stop cock connected to its upper side over the shorter glass tube, to allow the introduction of a wire when it was necessary to free the tube of air. The joints of the lead pipe and glass and metal tubes were made respectively of solder and gum-lac.

Manometer No. 5 was connected to the horizontal cylinder, or intermediate reservoir, near its outlet end. Manometer No. 4 was connected

Digitized by Google

to the experimental conduits quite near the horizontal cylinder. The difference between the readings of manometers Nos. 5 and 4, when small conduits were being used, gave the approximate loss of head due to the entrance of the water into the conduits from the horizontal cylinder, plus the head due to the velocity in the conduits. Manometer No. 3 was connected to the lead conduits 7.09 feet from the horizontal cylinder, and to the other conduits (with the exception of those of glass, the distance for which is not given by Mr. Darcy), from 13.68 to 19.29 feet from the horizontal cylinder. By an arrangement of cocks and connections, one 0.55 inch lead connection pipe served for both manometers Nos. 3 and 4. Manometer No. 2 was connected to all of the experimental conduits 164.05 feet from No. 3, with the exception of those of lead and glass, which were connected, respectively, 82.03 and 76.41 feet from No. 3. Manometer No. 1 was connected to all of the experimental conduits 164.05 feet from No. 2, with the exception of those of lead and glass, which were connected, respectively, 82.03 and 69.79 feet from No. 2. The distance from manometer No. 1 to the vertical outlet cylinders was upon the lead conduits about 16.41 feet, and upon the others, with the exception of the glass conduit, from 14.60 to 19.85 feet.

At first sight manometer No. 2 might not seem necessary, but it rendered great service during the experiments as a check upon Nos. 1 and 3, as well as in detecting, on several occasions, disturbances which would have materially interfered with the experiments, and which probably would not have been known but for this supplementary manometer. For instance, all things being equal, the difference in the observations of manometers Nos. 1 and 2 should be the same as the difference in the observation of Manometers Nos. 2 and 3. In actual practice, however, they were never exactly the same, as a slight variation in the mean diameter of the conduit, between either of these points, would cause a slight difference; but under these circumstances, as the difference was always in the same sense for all heads, the reason was readily understood. When these conditions were not fulfilled it was generally owing to leaks or accumulations of air.

There was also placed against the large outlet cylinder a graduated board with two glass tubes; one of these tubes was connected with the interior of the cylinder, and indicated the height of the water that it contained, and the other was connected to the experimental conduit near the cylinder, and indicated the head of the water moving in the conduit.

In all computations, the difference in the heights indicated by manometers Nos. 1 and 3, were always taken as the losses of head, and the lengths used were the distances between the points where these manometers were connected to the experimental conduits. The distance between Nos. 1 and 3 was for the glass conduit, 147.19 feet; for the lead conduits, 164.05 feet, and for all the other kinds of conduits,

Digitized by Google

328.09 feet. As the manometers were connected at the joints of the glass conduit, it was not possible to make the distances between Nos. 1 and 2 and Nos. 2 and 3 the same.

The absolute heads of the large reservoirs at Chaillot, or the circular sheet-iron tank, were used during the greater portion of the experiments. In order to vary the head for each experiment, while experimental conduits of large diameter were being used, the gate, or the stop cock of the branch conduits through which the water flowed that supplied the horizontal cylinder at the time, was throttled. This could not be done satisfactorily, however, when experiments were being made with small conduits. Two new apparatus were therefore constructed, for making the experiments with the glass and lead conduits, and with the wrought iron conduits 0.48 and 1.047 inches in diameter, by means of which reservoirs were formed of constant elevation, which furnished the different heads that were required.

The apparatus which was used with the wrought-iron experimental conduits consisted of a cast-iron column, composed of flanged pipes 9.84 inches in diameter. A lead pipe 0.55 inches in diameter, which was run up the side of the column and bent over the top, supplied the water while the experiments were being made; in the lower part of this lead pipe a cock served to shut off the water entirely, or to moderate the flow; the supply could also be regulated by means of a cock in the horizontal cylinder, to a branch of which the other end of the pipe was connected. Another lead pipe, 1.06 inches in diameter, was connected to a small branch at the bottom of the column, and ran up vertically by the side of the column; in this lead pipe, at convenient distances apart, there were branch openings, which were stopped with wooden plugs. A ladder was erected by the side of the column in order to manouver the plugs. When it was necessary to operate with the least head, the lower plug was removed, and by means of the cock in the lower part of the lead supply pipe, the arrival of the water was regulated in such a manner as to allow but a very small quantity to escape by the branch opening from which the plug had been removed, thereby maintaining a supply of water in the column at a constant level. In order to maintain a higher head, one of the upper plugs was removed and all the lower openings closed. The greatest head was obtained by closing all the branch openings and letting the water flow over the top of the column. The experimental conduits for which this apparatus was constructed, were connected to the lower part of the column and had a regulator cock near their origin. Manometer No. 5 was connected to the same branch as the 1.06-inch vertical lead pipe with the different branch openings.

The other apparatus, which was used with the lead and glass conduits, was constructed by taking the horizontal cylinder, 3.28 feet in diameter, and placing it vertically upon its semi-circular end, and con-

Digitized by Google

necting vertically to its other end a portion of the 9.84 inches cast-iron column that was used in making the experiments with the wrought-iron conduits. In order to create at will reservoirs of different levels, a sufficient length of the lead pipe with branch openings that had previously been connected to the cast iron column was connected to a branch at the lower end of the cylinder. Manometer No. 5 was connected to the same branch as the vertical lead pipe with branch openings. The supply of water was brought from the large conduits that formerly supplied the horizontal cylinder, by a lead pipe 1.61 inches in diameter, which was connected to a branch on the side of the cylinder opposite to the branch where manometer No. 5 and the vertical lead pipe with branch openings were connected; there was a stop cock on this supply pipe near its connection with the cylinder, to regulate the flow of water. The experimental conduits for which this apparatus was constructed were connected to a branch in the front of the cylinder, which was situated about half way between the branches where the supply pipe and vertical lead pipe were connected. Each experimental conduit had a stop cock near its junction with the cylinder to shut off the supply of water when necessary.

The experimental conduits of wrought-iron, lead and glass, for which the two apparatus that have just been described were constructed, discharged into the smallest of the two vertical outlet cylinders, and the general arrangement of the stop-cocks and manometers upon these conduits, with the exception of manometer No. 5, was the same as upon the other conduits.

General Observations Relating to Darcy's Experiments.

The pipes of each diameter were forced as close together as possible, in order to prevent the joints from interfering with the flow of the water.

Before commencing the experiments, the conduits were always tested under pressure, and if any leaks were discovered they were immediately repaired, even though they were of the most minute description.

After the conduits were in position the manometers were connected, and the locations of Nos. 1, 2 and 3 were determined with much care. After having connected manometer No. 2 at a convenient point near the manometer post, the proper distances were measured off above and below manometer No. 2, for fixing the positions of manometers Nos. 1 and 3. The measurements were made with an iron rod 16.41 feet in length.

Upon the cast-iron conduits 3.15 inches in diameter and above, the taps or stop-cocks of the 0.55-inch manometer connecting pipes were screwed into the cast-iron at right angles to the center of the conduits. These stop-cocks were filed at their lower ends to conform to the curvature of the interior of the conduits, beyond the surface of which they were not allowed to project.

Digitized by Google

Upon the conduits less than 3.15 inches in diameter, and upon the sheet-iron conduits coated with bitumen, the taps or stop-cocks of the manometer connecting pipes were soldered, above a hole from 0.08 to 0.12 inches in diameter.

The conduits, the stop-cocks, the joints, the air-cocks or vents, and the manometers, were all constantly under the most careful supervision; it was frequently necessary, however, notwithstanding the precautions taken, to make some of the experiments over a second time, on account of a small leak having been discovered at their conclusion, upon some portion of the apparatus.

Before commencing the experiments upon a conduit, the manometers were tested, by opening their stop-cocks and air vents, and putting the conduits under pressure Much difficulty was always experienced when the first experiment was commenced, in freeing the manometer pipes of air, and it was frequently necessary to let the water flow a long time under the greatest possible head in order to do so. During the experiments upon the conduit 0.48 inches in diameter, one entire day's work was entirely eliminated on this account, the difficulty being finally overcome by letting the water flow all night.

In order to be certain that the manometers were free from air, the difference of the readings of manometers Nos. 2 and 3, and Nos. 1 and 2, were compared. These differences, as has been previously mentioned, should be nearly equal, when the water was flowing in the conduits, if the manometers were working properly. Another test with the water at rest, was also made at times, during which the readings of all the manometers were exactly the same if they were entirely free from air.

Nearly all of the experiments were commenced with slight heads. As soon as the desired head was obtained, pins were put into the graduated plank to mark the heights of the manometers, and the water was allowed to flow long enough to acquire its normal regimen. When the water, or mercury, in the manometers became stationary, proceedings were taken to measure the discharge of the conduit.

The same person observed the heights of all the manometers when it was possible. While experiments were being made with low heads the observer was required to remain upon the manometer ladder the whole of the time, in order to have his eyes fixed constantly upon the manometers, and if a noticeable change in the heights, or an oscillation of extraordinary amplitude was observed, the experiment was commenced over again.

It was the duty of the person detailed to observe the heights of the manometers to watch carefully the conduits, the manometers and the other apparatus, in order to be assured that everything was working properly.

Considerable difficulty was experienced in making the experiments with high heads, as it was necessary to use mercurial manometers. Two

Digitized by Google

consecutive experiments were never made without it being necessary to repair one or more joints, and frequently repairs had to be made several times during the same experiment.

In order to verify some of the calculations which necessitated the use of the mercurial manometers, the water manometer No. 1 was always left open, and No. 2, also, when the heads were not too great.

Heights of water were thus obtained, with which the heights indicated by the mercurial manometers could be compared.

The highest accuracy was always maintained in measuring the discharge of the conduits, and the same watch graduated to seconds was nearly always used.

The time consumed in making each experiment ranged from 135 to 3 240 seconds; it was rarely less than 240 seconds, however.

Extraordinary care was taken in determining the mean diameter of the conduits.

The diameters of the conduits 0.48, 1.05 and 1.56 inches in diameter, were ascertained by means of the quantity of water that they were able to hold. The water was drawn from a reservoir of well-determined section, that was placed in an elevated position, and under which the conduits were vertically arranged in fractions of length.

At each operation account was kept of the lowering of the water in the reservoir, and the product of the sum of the successive lowerings by the section of the reservoir gave a cube, which, divided by the lengths of the conduits, gave the mean section, from which was obtained the mean diameter.

The conduits 0.55, 1.06 and 1.61 inches in diameter, of drawn lead, had a diameter perfectly well established.

The sheet iron conduits, coated with bitumen of 1.06 and 3.25 inches in diameter, and the glass conduit of 1.96 inches in diameter, were determined by the first process described.

When the diameters of certain conduits were too large for the above process to be easily carried out, or when it was a question of conduits lined with deposit, the diameters were ascertained by means of the total capacities of the conduits, the latter being in place. This proceeding, which was followed in determining the diameters of the sheet-iron conduit coated with bitumen, 7.72 inches in diameter, and the cast-iron conduits 1.43, 3.15 and 9.63 inches in diameter, was carried out in the following manner:

First.—The extreme pipe was first dismounted (that is to say, the pipe contiguous to the feeding horizontal cylinder at the head of the conduit, and towards which the conduit sloped), thus allowing the water to flow entirely out of the conduit.

During this operation the gate at the other extremity of the conduit near the vertical outlet cylinder, was kept closed, the cylinder having been previously filled with water.

Digitized by Google

Second.—The extremity of the conduit which had been disconnected from the horizontal feeding cylinder was plugged. At the extremity of this plug a curved pipe with a stop-cock was connected, in order to allow the air to escape, while the conduit was being filled from the vertical cylinder by the opening of the gate.

Care was taken to note the height of the water in the vertical cylinder before and after the filling.

A stop-cock to facilitate the escape of air was placed at the middle of the conduit.

Third.—In order to fill the conduit, the gate near the vertical outlet cylinder was opened, and two observers, one of whom was placed at the middle of the conduit, and the other at the extremity by the plug, closed the air-cocks as soon as the water appeared. The gate was then closed, and from the lowering of the water in the vertical cylinder the mean section of the conduit was determined, from which was calculated the mean diameter.

The diameters of the sheet-iron conduit coated with bitumen, 11.22 inches in diameter, and the cast-iron conduits 3.22, 5.39, 7.40, 11.69 and 19.69 inches in diameter, were obtained by direct measurements.

The diameters of the conduits lined with deposit, given in Table No. 1, are the diameters of the conduits after they were cleaned, and not the diameters used by Mr. Darcy, which took into consideration the thickness of the deposit.

Experiments of Dubuat.

"Principes d'Hydraulique," by Chevalier Dubuat, Paris, 1786.

Chevalier Dubuat, who was a celebrated French Civil Engineer, made quite a number of experiments with various sizes and lengths of pipe. The writer, however, has selected only the eighteen that were included in the fifty-one experiments used by Prony in determining his formula, which were made with a tin pipe 1.07 inches in diameter. Their tabular numbers are 106 to 123 inclusive.

The pipes used in making the eighteen experiments appear to have been laid straight, and discharged sometimes under water, and sometimes into the open air.

Experiments of Darrach.

"Transactions of the American Society of Civil Engineers," Vol. VII, 1878.

The fifty-three experiments were made under the direction of C. G. Darrach, M. Am. Soc. C. E., with six cast-iron pumping mains, one of 20, three of 30, and two of 36 inches in diameter, belonging to the Philadelphia Water Works. The pipes were coated with coal tar in the usual way, before being laid.

The experiments that were made with each main are described separately, and each description is preceded by the tabular numbers of the experiments to which it refers.

Digitized by Google

Pressure gauges were used in determining the head.

Twenty-inch.—Nos. 458 to 466 inclusive. This main was eleven years old when the experiments were made. It had one quarter turn. The flow was measured by the displacement of the pump plungers of a Worthington pumping engine, 5 per cent. having been deducted for slip. The presure gauge was connected with the pump.

Thirty-inch.—Nos. 473 to 485 inclusive. This main was about two years old when the experiments were made. It had four check-valves, the weight of which produced a pressure equivalent to 1.8 pounds per square inch, for which allowance was made in the frictional head given in Table No. 1. The flow was measured by the displacement of the pump plungers of a Cramp pumping engine, 5 per cent. having been deducted for slip. The pressure gauge was connected with the air chamber.

Thirty-inch.—Nos. 468 to 472 inclusive. This main was laid the same year that the experiments were made. It had one quarter turn. All the other curves had a radius of 25 feet. It also had two check-valves, the weight of which produced a pressure equivalent to 1.3 pounds per square inch, for which allowance was made in the frictional head given in Table No. 1. The flow was measured by the displacement of the pump plungers of a Worthington pumping engine, 5 per cent. having been deducted for slip.

Thirty-inch.—Nos. 486 to 495 inclusive. This main was about nine years old when the experiments were made. It had one curve of short radius. The flow was measured by the displacement of the pump plungers of a Worthington pumping engine, 5 per cent. having been deducted for slip. The pressure gauge was connected with the main outside of the check-valve.

Thirty-six-inch.—Nos. 497 to 502 inclusive. This main was seven years old when the experiments were made. All the curves had a radius of 25 feet. The flow was measured by the displacement of the pump plungers of a Simpson pumping engine.

Thirty-six-inch.—Nos. 503 to 512 inclusive. This main was seven years old when the experiments were made. All the curves had a radius of 25 feet, with one exception, when 90 degrees were turned with a T-pipe. The flow was measured by the displacement of the pump plungers of a Worthington pumping engine, 5 per cent. having been deducted for slip. The pressure gauge was connected with the stand-pipe.

Experiments of the Edinburgh Water Company.

"*Excerpt Minutes of the Proceedings of the Institution of Civil Engineers,*" *Vol. XIV, Session* 1854–55.

The three experiments were mentioned by Mr. James Leslie, M. Inst. C. E., in a paper on the flow of water through pipes, conduits, etc. Their tabular numbers are 415 to 417 inclusive.

Digitized by Google

They were made with the Clointon cast-iron main between Clubbie-Dean Reservoir and Castle Hill. This main was 16 inches in diameter and eight or nine years old.

One experiment was made with the total length of the pipe, 29 580 feet; one with a length of 25 765 feet, between Torduff Cistern and Castle Hill; and one with a length of 3 815 feet, between Clubbie-Dean Reservoir and Torduff Cistern. In the first instance fifteen observations were made, in the second twenty-five, and in the third one.

Experiments of Fanning.

"*A Practical Treatise on Hydraulic and Water Supply Engineering,*" *by James T. Fanning, C. E. New York*, 1882.

The twelve experiments were made with a wrought-iron cement-lined pipe, 20 inches in diameter, and 8 171 feet long. Mr. Fanning does not mention the manner in which they were conducted. Their tabular numbers are 437 to 448, inclusive.

Experiments of Gale.

"*Transactions Institution of Engineers in Scotland,*" *Vol. XII*, 1869.

The experiment was made under the direction of James M. Gale, M. Inst. C. E., with a cast-iron main, 48 inches in diameter, and 5 280 feet long, that conducted a supply of water from Loch Katrine to the City of Glasgow. Its tabular number is 517.

The writer is indebted to Mr. James Forrest, Secretary of the Institution of Civil Engineers, for the following additional information: "Mr. Gale informs me that the pipes referred to in his paper had been at work for eight years prior to the date of the observations. Before being laid they were coated by the late Dr. R. A. Smith's patent process, and this coating had been very little affected by rust when the observations were made. He has recently seen some of these pipes" (August, 1884), "and finds that the tubercles inside are increasing in size and number, and he has no doubt that the discharge is falling off. The quantity of water delivered was measured over a weir 40 feet wide, in four bays of 10 feet wide each. The plates were of cast-iron brought to a thin edge. The formula used was cubic feet per minute $= 214\ A \sqrt{h}$."

Experiment of Greene.

"*The Brooklyn Water Works and Sewers,*" *by James P. Kirkwood. New York.* 1867.

This experiment was made by General George S. Greene, with a cast-iron main, 36 inches in diameter, and 11 217 feet long, belonging to the Brooklyn Water Works. Its tabular number is 496.

Digitized by Google

This main connected the receiving with the distributing reservoir. It was heavily tuberculated, and had three-quarter circle curves of 90 feet radius each.

The head was ascertained by leveling between the reservoir surfaces, and the flow was measured by the lowering of the water in the upper or feeding reservoir.

Experiment of Hodson.

"*Hydraulic Tables, Co-efficients and Formulas,*" *by John Neville, C. E., London,* 1860–61.

Mr. Neville states that the results of this experiment, which was made with a pipe 1 inch in diameter and 100 feet long, was given to him by Mr. Hodson, of Lincoln.

As Mr. Neville used 0.352 as the co-efficient of influx in connection with this experiment, the writer has done the same.

Experiments of Jardine.

"*Brewster's Encyclopædia.*"

The experiment was made with a lead pipe 4¼ inches in diameter, and 14 930 feet long, belonging to the Edinburgh Water Works. Its tabular number is 308.

Mr. Jardine states that the flow through the pipe was 11¼ cubic feet per minute, for the years 1738–42. The velocity given in Table No. 1 is computed from these data.

Experiments of Leslie.

"*Excerpt Minutes of Proceedings of the Institution of Civil Engineers,*" *Vol. XIV, Session,* 1854–55.

The experiments were made under the direction of James Leslie, M. Inst. C. E.

Mr. Leslie made sixty-four experiments with new lead pipes 2½, 1¼ and 1½ inches in diameter. The writer has selected, and included in Table No. 1, only twenty of the thirty-eight experiments that were made with pipes 2½ inches in diameter, ranging in length from 100 to 1 086 feet; these experiments were simply selected so as to include velocities from 0.22 to 6.91 feet per second, the latter velocity being the highest that was obtained with these lengths of pipe. Their tabular numbers are 250 to 269 inclusive.

The experiments were commenced with a pipe 1 086 feet long; "the pipe was afterwards cut, successively, to the lengths of 540, 270, 100, 25 and 10 feet."

"The pipe was coiled nearly in a circle of 90 feet in diameter, and when of its integral or full length it made four revolutions.

Digitized by Google

Each upward curve was tapped by a small gimlet hole, into which a wooden plug was inserted; these were all taken out frequently to allow the air to escape, and were kept out until only water, free from air, issued from the orifice. The ends of the pipe, in each case, were inserted into the sides of two cisterns, each measuring 2 feet 6 inches by 1 foot 3 inches, and 2 feet deep, and were about 22 inches below the surface of the water. In order to be perfectly assured that there could be no mistake in the levels, the two cisterns were placed quite close together, and the head was measured by the difference of the respective surfaces of the water." The head could be varied at will by raising or lowering the upper cistern, which was supplied with water by a pipe that was carried up over the top, thence nearly to the bottom of the cistern. The water overflowed from the lower or outlet cistern, which was fixed, into a measuring box.

"The pipes were carefully joined and soldered, so as to have no internal obstruction, with the exception of one joint, where it was discovered, that by some accident, a little solder had got inside and caused a slight obstruction. The extent of it measured only about one twenty-fifth part of the area of the pipe, and would, it is imagined, be so trifling as not to appreciably affect the flow."

Experiments of Lampe.

"Der Civilingenieur." Vol. XIX, 1873; *and "Hydraulics," by Hamilton Smith, Jr., M. Am. Soc. C. E. New York*, 1886.

The four experiments were made with a 16-inch main, by Professor Dr. C. J. Lampe, an eminent German scientist. Their tabular numbers are 418 to 421 inclusive.

They are described by Mr. Smith as follows: "The pipe experimented upon was a cast-iron conduit, which conveyed by gravity a supply of spring water to the Town of Danzig. It had a total length of 46 352 feet; the lower 9 040 feet in length had a considerably steeper inclination than the upper portion, so there were two hydraulic grade lines; the four experiments, however, were made upon the upper portion of the pipe. The pipe was laid in 1869; it had three curves, each of 10.3 feet radius, and a number of very easy curves as it followed the general contour of the ground. The pipe was coated with a patent varnish which did not appreciably diminish its section. Examination showed that from 1869 to 1871 the character of the inner surface had very slightly changed; the only material adhering to the surface in 1871 could be readily removed by rubbing with the finger, there being no signs of rust. The joints were 12 feet in length, united by lead and hemp packings. There were twenty-six air-cocks attached to the pipe along its course."

Digitized by Google

"The mean velocity was ascertained by measuring the discharge * * * in a masonry reservoir situated at the outlet end of the pipe.

"The pressures were determined by connecting a quicksilver manometer, first with one of the air-cocks, and then with another; these pressure determinations were not synchronous with the measurement; but, in some instances, several days apart. As in none of the four experiments was the pipe filled at its inlet, this lack of synchronism appears to form a dangerous source of error, the pipe being fed by the flow from springs, whose discharge must necessarily have been more or less irregular. Dr. Lampe, however, states to us in an explanatory letter, that he is satisfied no serious error could have arisen from this cause, there having been, directly preceding or during these intervals, no rains of consequence to notably affect the flow from the feeding springs; subsequent readings of the manometer, shortly after these intervals, also verified the constancy of flow. He considers that errors from this source will not change the given results more than one-half of 1 per cent.

"Another source of error in these experiments arises from the fact that the pipe had two hydraulic grade lines. Although the several pressures were all determined above the anticlinal point at which these lines united, still the condition of the pipe was more or less unfavorable to extreme accuracy of observation, as the sucking or siphon action as the water passed over the summit could not have been perfectly regular, as doubtless the amount of air accumulating at this summit varied slightly from time to time; the consequent intermittant sucking action of the water as it flowed below this summit, hence may have appreciably affected the nearest piezometers."

Experiments of Neville.

"*Hydraulic Tables Co-efficients and Formulas*," *by John Neville, C. E. London*, 1860–61.

Mr. Neville states that the two experiments were the mean results of several experiments that were made by him with great care, with a pipe 1.02 inches in diameter and 9.29 and 19.20 feet in length. Their tabular numbers are 59 and 60.

Mr. Neville also ascertained that the co-efficient of contraction of the inlet end of the same pipe was 0.860, which corresponds to a co-efficient of influx of 0.352. This value was used by the writer in calculating the co-efficients of friction from these experiments.

Experiments of Provis.

"*Excerpt Minutes of Proceedings of the Institution of Civil Engineers*." *Vol.* II, 1838.

The experiments were made under the direction of W. A. Provis, M. Inst. C. E., with lead pipes 1.5 inches in diameter. Thirty-seven of

Digitized by Google

them have been selected by the writer. Their tabular numbers are 175 to 211 inclusive.

The pipes were drawn in 15 foot lengths, and soldered together with care, so as to avoid as far as possible any interior obstructions. Into the upper end of the pipe was inserted a stop-cock of similar bore, and from this cock the length of the pipe was measured; the opposite end of the cock was inserted into a cistern in which the surface of the water was maintained at a constant elevation during each experiment. The flow of water through the pipes, which discharged into the open air, was measured in a tank having a capacity of four cubic feet.

Experiments were made with pipes 100, 80, 60, 40 and 20 feet long.

The co-efficient of influx was not ascertained by Mr. Provis, and the writer, after having calculated two independent sets of experimental co-efficients of friction from the experiments that were made with the pipes 100, 80 and 60 feet long, by using 0.97 as a co-efficient of influx in one instance, and 0.505 in the other, finally determined to work out co-efficients of influx and loss from the experiments themselves. This was done by taking the data of the experiments made with the pipe 20 feet long, and working out co-efficients of influx and loss for different velocities of flow for this length of pipe. The co-efficient of influx and loss thus determined were used in calculating the experimental co-efficients of friction of the experiments which were made with pipes 100, 80 and 60 feet long, as a slight variation in its value one way or the other would not materially affect the results obtained from these experiments, on account of the long lengths of pipe used, and their low velocities of flow; with the experiments made with the pipe 40 feet long, however, there seems to be a question of doubt, consequently they were not considered by the writer. The values of the experimental co-efficients of friction obtained by using this latter co-efficient of influx and loss, were in the majority of cases more than was obtained by using as a co-efficient of influx 0.97, and less than was obtained by using 0.505. The lengths and heads of the pipes given in Table No. 1 are therefore less than those given by Mr. Provis; the lengths by 20 feet, and the heads by an amount equivalent to the loss of head due to contraction at the entrance of the stop-cock, plus the loss of head due to friction in the stop-cock, and in a length of pipe 20 feet long.

Experiments of Robison.

"*Encyclopædia Britannica.*"

The experiment is mentioned in an article on Rivers by Dr. John Robison, as having been made with a pipe 2 inches in diameter, and 3 300 feet long, which was part of a compound pipe that was used to convey water from a spring into the town of Haddington. The nature of the pipe and the manner in which the experiment was conducted is not given. Its tabular number is 237.

Digitized by Google

Experiments of Rennie.

"Traité d'Hydraulique," by J. F. D'Aubuisson de Voisins.

The two experiments were made with a lead pipe 0.5 inch in diameter and fifteen feet long. The manner in which they were conducted is not mentioned. Their tabular numbers are 28 and 29.

Experiments of Simpson.

"Excerpt Minutes of the Proceedings of the Institution of Civil Engineers." Vol. XIV, Session 1854–55.

The eighteen experiments were made under the direction of the late James Simpson, M. Inst. C. E., with pipes 12, 19 and 30 inches in diameter. Their tabular numbers are: 12 inch, 390 to 396 inclusive, 19 inch, 423 to 427 inclusive, 30 inch, 467.

Table No. I contains substantially all that Mr. Simpson mentioned in regard to the experiments. The following additional information, however, was furnished to the writer, in reply to some inquiries, by Mr. James Forrest, Secretary of the Institution of Civil Engineers:

"Mr. Arthur Telford Simpson, one of the sons of the late Mr. James Simpson, informs me that he cannot find the exact dates of the several experiments on the flow of water through pipes to which his father referred in the discussion at their institution in 1855, but he gives the following statement of the time when the mains were laid, which will serve to show that they could not have been in use a very long time."

Diameter in inches.	Place.	When laid.
12	Brixton to Streatham	1848
19	Belvedere Road to Brixton	1841
12	Liverpool	....
12	Carlisle	1850
30	Ditton to Brixton	1852

Experiments of Smeaton.

"Practical and Experimental Researches in Hydraulics," by R. A. Peacock, C.E.

The sixteen experiments were made by John Smeaton, an eminent English Civil Engineer, with a pipe 1¼ inches in diameter, and 100 feet long. The manner in which they were conducted and the nature of the pipe that was used are not mentioned.

Their tabular numbers are 124 to 139 inclusive.

Digitized by Google

Experiments of Smith.

"*Transactions of the American Society of Civil Engineers.*" *Vol. XII*, 1883.

The experiments were made under the direction of Hamilton Smith, Jr., M. Am. Soc. C. E., with pipes of various kinds, lengths and diameters.

The writer has selected, and included in Table No. 1, only forty-two of the seventy-one experiments that were made by Mr. Smith. Twenty-seven of the remaining twenty-nine were omitted on account of having been made with small pipes, less than 64 feet in length, which had funnel shaped inlets, and two others on account of there being a question of doubt in regard to the discharge in one instance, and the diameter of the pipe used in the other.

The fourty-two experiments were made with pipes of 0.502, 0.628, 0.746, 1.054, 1.26, 10.93, 12.67, 14.76 and 16.99 inches in diameter.

The experiments are described in divisions, which are preceded by the tabular numbers of the experiments, to which they refer.

Nos. 365 to 369, 405 to 408 and 409 to 414, inclusive (10.93, 12.67 and 14.76 inches), "were made at North Bloomfield, California, with their pipes laid side by side across a ravine about 100 feet lower than the penstock. They were made of single-riveted No. 14 sheet-iron; the rivet heads had worn pretty smooth, and no deduction for them was made in computing the areas. The pipes had been carefully coated when first laid with a mixture of coal tar asphaltum, and their interior surfaces at the time of these measurements were quite smooth. They were put together in slightly conical joints, the greatest variation in diameter being about ¼ of an inch. The main joints were some 20 feet in length, put together stove-pipe fashion, as is always done in the California hydraulic mines. The various heads were obtained by adding to the length of the pipes at their outlets."

"The heads, lengths and mean diameters are given with exactness. The amount of discharge was measured by running the water over an iron weir, the co-efficients of whose discharge, with varying depths, had been carefully determined by absolute measurement, but at another point, and under conditions not exactly similar. This gauging was done with all the care practicable, the height of the water being measured with Boyden hook gauge, reading to 0.001 of an inch. The limit of possible error in these experiments is probably not over 2 per cent."

Nos. 407 and 408, "however, are somewhat unreliable, owing to the stoppage of a stone passing through the pipe."

"The co-efficient of contraction is assumed at 1 in these pipes, as each of them had a short funnel-shaped inlet, which was included in the measurement of their lengths."

In Nos. 369, 408 and 414, "the discharge was into a pool of water at the outlet box; in the other twelve the discharge was into the air."

Digitized by Google

"There were two angles in the line of these pipes, one of 9 degrees and the other of 11 degrees; for the remainder of the distance they were nearly straight; in the computations no deduction was made for these bends."

Nos. 30 to 33, 41 to 46, 47 to 50, 78 to 84 and 140 to 144, inclusive (0.502, 0.628, 0.746, 1.05 and 1.26 inch), were made at New Almaden, California. "All these experiments were with straight pipes, and were made with considerable accuracy."

"The mean diameters were computed by the weight of water contained in the pipes, taken by the sum of weights in the several joints, which varied slightly from the mean of the end diameters as directly measured."

"The discharge was determined by filling a tank of known size."

"The iron gas pipes were joined by their usual couplings. The ends of the glass pipes were smoothly ground, and a water-tight joint made by an outside rubber band."

"The wooden pipe was joined by usual plug couplings."

Nos. 78 to 84, inclusive, were made with a pipe that was new and in good order.

"The glass tubes were new and unscratched, of French manufacture, and somewhat conical."

"The measuring tank held 15.2 cubic feet, and the times were determinated to one-fifth of a second."

No. 422 (16.99 inch) "was with a new inverted siphon of double riveted sheet-iron pipe, with a pressure of about 800 feet at the lowest point, with a maximum tensile strain of 16 500 pounds per square inch, and with most of the joints made by sleeves with lead packing. It was coated with the usual mixture of coal-tar and asphaltum."

"No deduction was made for rivet heads, which for over one-half the length doubtless formed noteworthy obstructions, as the pipe was of comparatively small diameter—17 inches."

"The mean diameter head and length are given with exactness. The discharge was measured first by the flow over a weir, and afterwards through standard apertures."

"In all the experiments the actual or total head was either the difference in elevation between the surface of the water in the pen-stock and that in the outlet tank, or the difference between the water in the pen-stock and the center of the escaping jet, where the pipe discharged into the air." Experiments 30 to 33, 41 to 46, 47 to 50, 78 to 84 and 140 to 144, inclusive "were made under the latter condition."

Experiments of Stearns.

"*Transactions of the American Society of Civil Engineers.*" *Vol. XIV*, 1885.

The four experiments were made by F. P. Stearns, M. Am. Soc. C. E., with a pipe 48 inches in diameter, and 17 472 feet long, belonging

Digitized by Google

to the Boston Water Works. Their tabular numbers are 513 to 516, inclusive.

"The pipe was on the line of the Sudbury conduit. It was a cast-iron pipe coated with Dr. Angus Smith's coal-tar preparation. The changes in direction were made by two vertical curves, one of 500 and the other of 1 170 feet radius. The mean pressure on the pipes during the experiments was 41 feet. The pipes had been laid three years, and had been in use two, when the experiments were made, yet the tar-coating presented nearly as good a surface as when it was new. The volume flowing was measured at a weir about 10 miles distant up the conduit from the pipe, and to this measurment $\frac{3}{10}$ of a cubic foot per second was added for filtration into the conduit below the weir. The amount of the filtration was determined from fairly good data; but, even if somewhat inaccurate, it has little importance, as its whole effect was less than 1 per cent. The weir used was 19 feet long, and had previously been tested by the actual measurement of the water passing over it. The loss of head in a length about 60 feet shorter than the whole length of the pipe was measured. This method was adopted to avoid including in the measurement the loss of head at the entrance of the pipe, the gain or loss at the exit, the effect of two rather sharp curves (30 feet radius), near the ends of the pipe, and several other disturbing causes. The apparatus used for taking the heads was as follows: In each of the pipe chambers, at either end of the pipe, there was a float gauge designed for permanent use in measuring the height of the water in the chambers. This consisted of a vertical iron cylinder, 12 inches in diameter, plugged at the bottom; in this cylinder was a hollow brass float, with a suitably guided stem, carrying an index up and down the face of a graduated scale. Water was admitted to the float cylinder through a small pipe leading from the center of the pipe chamber, the flow being controlled by a stop-cock. Connected with the float cylinder was a small brass cup, having a level top of known elevation. By filling the cylinder until the cup was even full, the position of the index was adjusted. For the purpose of these experiments, the small pipes leading from the float cylinders were extended into the ends of the 48-inch pipe, along its bottom about 33 feet. The last 7 feet of each of these small pipes was a smooth, straight brass tube, with several holes drilled in its top, and with its end plugged."

The heights of the gauges were carefully tested, and precautions taken to expel all air from the small pipes leading to them.

Mr. Stearns was not able to conduct the experiments in person, but he gave detailed instructions as to the work to be done, to two experienced assistants.

Digitized by Google

Experiments of Weisbach.

"*The Mechanics of Engineering and of the Construction of Machines,*" *by Julius Weisbach, Ph.D. Translated from the German edition by Eckley B. Coxe, A.M. New York,* 1872.

The experiments were made with pipes 0.409, 0.563, 0.972 and 1.27 inches in diameter.

Professor Weisbach only mentions the experimental co-efficients of friction (ζ) computed from the twenty-one experiments, and the nature and diameters of the pipes used in making the experiments. Their tabular numbers are, 0.409-inch and 0.563-inch, 1 to 14, inclusive; 0.972-inch, 52 to 57, inclusive, and 1.27-inch, 145.

Experiments of Weston.

The five experiments were made by the writer during the years 1876 and 1877 with pipes 0.97 and 6 inches in diameter, belonging to the Providence Water Works.

The experiments that were made with each size of pipe are described separately, and each description is preceded by the tabular number or numbers of the experiment or experiments to which they refer.

Six-inch.—Nos. 325 to 328 inclusive. An ordinary straight cast-iron pipe with socket joints, which were calked with lead and yarn in the usual way, was used in making these experiments. The pipe had been coated when new by Dr. Angus Smith's process, and at the time the experiments were made it had been in service four years.

The supply of water was taken from a main 30 inches in diameter, which was connected with the upper end of the pipe.

Two very accurate hydraulic pressure gauges were used to determine the head; the first was connected with the 6-inch pipe 500 feet from the 30-inch main, and the second 1 170 feet from the first, and 410 from the lower end of the pipe where the discharge took place. The total length of the pipe was 2 090 feet. The difference in the readings of the two gauges, allowance having been made for their elevation, was taken as the loss of head due to friction for a length of 1 170 feet.

The lower end of the pipe was connected with an 8-inch flush hydrant, which had a "chuck" with four lines of 2½-inch rubber hose about 100 feet long connected to it. The flow in the 6-inch pipe was regulated by opening and closing the valves of the "chuck." The first experiment was made with one valve open, the second with two, the third with three and the fourth with four. The discharge was ascertained by measuring the water flowing in the hose, which was done by connecting a hydraulic pressure gauge 1 foot from the hydrant to the line or lines of hose in use, and calculating the discharge from the pressure, a co-efficient of discharge having been previously determined

Digitized by Google

by measuring the flow under different pressures, in a large tank arranged expressly for the purpose.*

Considerable time was devoted to making the observations, and a sufficient period was allowed for the water to get in train after each experiment, before commencing the next. The pipe had several side connections which were carefully closed before the experiments were begun.

0.969-*inch.*—No. 51. A new wrought-iron pipe, which had a layer of tin drawn through it for a lining, was used in making this experiment.

The lengths were coupled together like ordinary gas pipe, with the exception that sheet tin bushings were fitted into the pipe at the joints to prevent any corrosion from penetrating into the interior, that might take place at the ends of the lengths where the iron was not covered with tin.

The joints were carefully made, and the interior surface of the pipe was practically as smooth as the interior surface of an ordinary lead pipe.

The pipe was 781.83 feet long, was laid nearly straight and was connected by a $\frac{5}{8}$-inch tap with a main 36 inches in diameter, which was directly supplied with water from a large reservoir.

The diameter of the pipe was determined by measuring the diameters of a number of lengths.

The difference between the surface of the water in the reservoir, and the center of the outlet end of the pipe which discharged into the open air, was taken as the head.

The discharge was very accurately measured in a large tank.

Three independent experiments were actually made, but as their results are so nearly alike, their average only is given in Table No. 1.

The co-efficient of influx and loss of head in the $\frac{5}{8}$-inch tap were ascertained by experimenting with a short piece of pipe connected to the tap, previous to making the experiment with the long pipe.

A NEW FORMULA FOR THE FLOW OF WATER IN PIPES HAVING VERY SMOOTH INTERIOR SIDES FROM $\frac{1}{4}$ TO $3\frac{1}{4}$ INCHES IN DIAMETER.

Before proceeding any further, it may be well to repeat, that, for reasons already mentioned, the basis of investigation which was adopted by the writer at the commencement of his researches, is the co-efficient of friction ζ, which is a factor of the equation,

$$h_f = \zeta \frac{l}{d} \frac{v^2}{2g}.$$

Consequently, the investigations which will now be described, as well as

* Transactions Am. Soc. C. E., Vol. XIII, 1884. Weston on the Flow of Water through Rubber Hose.

Digitized by Google

those which will be mentioned hereafter, have been entirely devoted to the determination of a formula for the co-efficient of friction ζ.

It was found, by consulting Table No. 1, that there were two hundred and thirty-eight experiments that had been made with twenty-seven different pipes having very smooth interior sides; one hundred and ninety-one of these experiments were made with pipes of lead, glass, zinc, tin, and sheet-iron coated with bitumen from 0.41 inch to 3.25 inches in diameter; eighteen that had been made with pipes of sheet-iron coated with bitumen 7.72 and 11.22 inches in diameter; thirteen that had been made with new gas pipe from 0.63 to 1.05 inches in diameter; and sixteen that had been made with a pipe 1.25 inches in diameter, the nature of which the writer has not been able to ascertain, although it evidently had smooth interior sides.

The last twenty-nine experiments mentioned, that were made with iron pipe, etc., were not given the same weight in the investigations as those that had been made with pipes which were known to have very smooth interior sides, but were more especially used for the purpose of substantiating the laws and results obtained with the latter.

Then, as pipes of 7.72 and 11.22 inches in diameter having very smooth interior sides are rarely used, and as the laws that seemed to apply to the results of the experiments that were made with the pipes of these diameters were slightly at variance with the laws that applied to the others, it was decided to consider only the experiments that were made with the pipes of the smaller sizes, in constructing the new formula, and to limit its application so as to include only pipes from 0.40 to 3.50 inches in diameter. (An independent formula was also determined by the writer for the two pipes, 7.72 and 11.22 inches in diameter, which will be mentioned hereafter.)

The diagrams which refer to this formula are numbered from 1 to 14, the first six being devoted to illustrating the method by which the formula was constructed, and the last eight to its comparison after completion.

For convenience of investigation, when it could be done without affecting the results sought after before reaching the fourth decimal place, the larger part of the experimental results that were obtained with pipes having diameters which were very nearly alike, were classified in each instance, under a common nominal diameter, as will be seen upon the diagrams.

Digitized by Google

The nominal diameters of the pipes that were used in making the experiments, the results of which were employed in constructing the formula, etc., the numbers that the experiments are given in Table No. 1, and the numbers of the diagrams upon which the results are platted, are as follows:

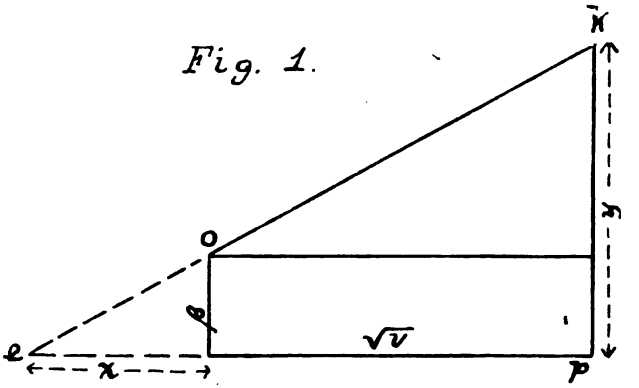
0.5-inch: Tabular numbers 1 to 14, 28 to 29, 34 to 46. Diagrams Nos. 1 and 7.
0.75-inch: Tabular numbers 47 to 50. Diagram No. 8.
1.0-inch: Tabular numbers 51 to 60, 78 to 123. Diagrams Nos. 2 and 9.
1.25-inch: Tabular numbers 124 to 139 and 145. Diagrams Nos. 3 and 10.
1.50-inch: Tabular numbers 146 to 160, 175 to 211, 224 to 230. Diagrams Nos. 3 and 11.
2.13-inch: Tabular numbers 231 to 249. Diagrams Nos. 4 and 12.
2.50-inch: Tabular numbers 250 to 269. Diagrams Nos. 4 and 13.
3.25-inch: Tabular numbers 296 to 307. Diagrams Nos. 5 and 14.

The results of any of the experiments that were made with velocities of less than 1 foot per second were not used in the actual construction of the formula, however, owing to the influence that minute errors of observation have upon results obtained with low velocities. For example, an error of 0.01 foot in observing the loss of head, in making an experiment with a pipe 1 inch in diameter and 100 feet long, when the velocity of flow was 0.25 feet per second, would affect the value of the co-efficient of friction to a greater extent than would an error of 1.5 feet in observing the loss of head, in making an experiment with the same pipe when the velocity of flow was 10 feet per second. The results shown on the diagrams from No. 1 to No. 5, that were obtained with velocities less than 1 foot per second, were platted after the formula was completed for the purpose of verification and comparison.

The first steps that were taken in the construction of the formula were to determine the conditions upon which the co-efficient of friction ζ was dependent, and after considerable preliminary platting, computation and study, it was found to be dependent, in this instance, upon both the diameter and the square root of the velocity.

The process by which the formula was finally developed, after the laws that applied to it had been ascertained by a method somewhat similar, is illustrated by Figs. 1 and 2, and upon the diagrams from No. 1 to No. 6.

Digitized by Google



An independent formula $\zeta = \alpha + \frac{\beta}{\sqrt{v}}$ was first determined for each diameter of pipe that the experimental data embraced.

The numerical values of α and β were deduced graphically in the following manner by the aid of the expression $\zeta \sqrt{v}$, which is the equation of the straight line $o\ k$, Fig. 1 (and the average lines shown upon the diagrams from No. 1 to No. 5), whose abscissæ are the values of $\sqrt{v}$, and whose ordinates are the values of $\zeta \sqrt{v}$.

A line was drawn upon the lower part of a large sheet of paper, with a sharp pointed lead pencil, as an axis of abscissæ, and another line drawn in the same way at right angles to it, at its left extremity, as an axis of ordinates.

Then at a scale of 1 inch to 4 feet the square roots of the velocities $(\sqrt{v})$ that were used in computing the experimental co-efficients of friction (ζ) were platted as abscissæ, and the experimental co-efficients of friction multiplied by the square roots of their respective velocities $(\zeta \sqrt{v})$ platted as ordinates, at such a scale that four decimal places could be easily read.

At the upper extremity of each ordinate a small circle or symbol was drawn in ink, as shown on the diagrams from No. 1 to No. 5, in order to render it conspicuous, and to designate the results obtained from the experiments of each authority.

If all of the minute errors could have been detected, that were, presumably, made in conducting the experiments from which the platted results were obtained, and if the conditions relative to the diameters and the interior sides of the different pipes that were used had been pre-

Digitized by Google

cisely the same, and if the laws upon which the formulas were based were absolutely correct, all of the circles or symbols upon each diagram from No. 1 to No. 5, would be in a true inclined line. As it is not possible to realize all of these theoretical conditions in actual practice, however, it was always necessary to draw an average line through the circles or symbols, as shown upon the diagrams from No. 1 to No. 5. This line was located by the aid of a piece of fine thread, and its position was generally such a very difficult matter to determine, that it was often not satisfactorily accomplished until after a number of trials.

When the average line was drawn, it was extended through the axis of ordinates to a prolongation of the axis of abscissæ, and the distance x, Fig. 1, scaled on this prolongation, from the point where the average line intersects it, to the axis of ordinates. A line was also drawn with a fine pointed lead pencil as an ordinate, from a point corresponding to the square root of some high velocity, at the extreme right of the axis of abscissæ, and the distance y, Fig. 1, between the axis of abscissæ and the average line, scaled upon it. Then the value of β was ascertained by scaling the distance on the axis of ordinates between the axis of abscissæ and the average line, Fig. 1, and the value of α, which is the tangent of the angle $k\ e\ p$, determined thus,

$$\alpha = \frac{y - \beta}{\sqrt{v}} = \frac{\beta}{x}$$

and the expression for the co-efficient of friction ζ, becomes for each diameter of pipe,

$$\zeta = \frac{y - \beta}{\sqrt{v}} + \frac{\beta}{\sqrt{v}} = \frac{\beta}{x} + \frac{\beta}{\sqrt{v}} = \alpha + \frac{\beta}{\sqrt{v}}.$$

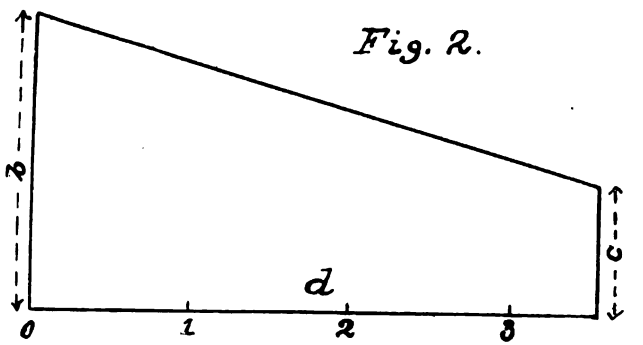
For example, let

$$\beta = 0.0283,\ y = 0.0622,\ \sqrt{v} = 3,\ x = 1.347, \text{ and } \zeta = 0.0113 + \frac{0.0283}{\sqrt{v}}.$$

When a formula for each diameter had been determined in the manner just described, it was found by taking the values of α and β, and platting them in a similar manner, and with the same degree of care, the diameters as abscissæ, and the corresponding values of α and β as ordinates, the extreme upper points of which were enclosed in circles, and drawing an average line from the axis of ordinates through the circles representing each as shown upon Diagram No. 6, that α was constant for all diameters and velocities, and that β decreased slightly as the diameters increased. The value of α, being constant, was ascer-

Digitized by Google

tained by simply scaling the distance, which was found to be 0.0126, between the axis of abscissæ and the average line, they being parallel in this instance. The value of the diametric co-efficient, which we will call z, upon which β is dependent, was determined by measuring the distance b, Fig. 2, on the axis of ordinates between the axis of abscissæ



and the average line, and by drawing a line as an ordinate from the abscissa representing the greatest diameter, and scaling upon it the distance c, Fig. 2, between the axis of abscissæ and the average line, and using their values in this form,

$$z = \frac{b-c}{d} = \frac{0.0315-0.014}{3.5} = 0.005$$

and the expression for β becomes

$$\beta = b-d\,z,$$

and the formula for the co-efficient of friction ζ, for all diameters from 0.40 to 3.5 inches, and for all velocities

$$\zeta = 0.0126 + \frac{0.0315-d\ 0.005}{\sqrt{v}}.$$

On account of the numerical values of the diameters having been used in inches in determining this formula, it can only be used in its present form when the diameter, d, is expressed in inches. (This is the only instance, with the exception of the cases when the same formula is shown upon the diagrams from No. 6 to No. 14, that any formula contained in this paper is given in measures other than feet.)

Digitized by Google

The formula therefore becomes, for measures in English feet,

$$\zeta = 0.0126 + \frac{0.0315 - d\,0.06}{\sqrt{v}}$$

$$\text{and } hf = \left(0.0126 + \frac{0.0315 - d\,0.06}{\sqrt{v}}\right) \frac{\iota}{d} \frac{v^2}{2g}.$$

Diagrams from No. 7 to No. 14 show a comparison of this formula with the original experimental results from which it was derived, as well as with a number of other experimental results that were not used in constructing the formula.

Co-efficients of friction (ζ), that have been worked out from several well-known formulas for the flow of water through pipes, by different authorities are also platted for comparison on the diagrams from No. 7 to No. 15.

The equations for computing these last mentioned co-efficients of friction, as arranged by the writer, are as follows:

From the formula of Prony,

$$\zeta = 0.02733 + \frac{0.004464}{v}.$$

From the formula of Eytelwein,

$$\zeta = 0.0220 + \frac{0.005754}{v}.$$

From the formula of Darcy,

$$\zeta = 0.01989 + \frac{0.001666}{d}.$$

From the formula of Weisbach,

$$\zeta = 0.01439 + \frac{0.01704}{\sqrt{v}}.$$

Equations for the flow of water in pipes, in which the co-efficient of friction ζ, worked out by this formula, is used, will not admit of a direct solution, but the co-efficients should be first determined for different values of the velocity and tabulated, after which the true value of the velocity can be determined by finding an approximate value, and thence taking out the corresponding co-efficient from the table, which does not vary to any considerable extent for small changes of velocity.

The writer would have been much better satisfied had he been able to construct a formula that could be more easily solved than the one he has presented, but as his principal object was accuracy and not simplicity, he could not do this and conform satisfactorily to the experimental results.

Digitized by Google

The new formula will not be further discussed, as its merits or demerits are plainly set forth upon the diagrams upon which it is platted.

The following table gives the co-efficients of friction (ζ), for pipes having very smooth interior sides from 0.5 to 3.5 inches in diameter, and for velocities from 0.10 to 50 feet per second, calculated by the formula.

$$\zeta = 0.0126 + \frac{0.0315 - d\,0.06}{\sqrt{v}}.$$

CO-EFFICIENT OF FRICTION ζ.

Velocity in feet per second.	DIAMETER OF PIPES IN FEET AND INCHES.									
	0.0417 ½	0.0521 ⅝	0.0625 ¾	0.0833 1	0.1042 1¼	0.1250 1½	0.1667 2	0.2083 2½	0.2500 3	0.2917 3½
0.10	0.1043	0.1023	0.1004	0.0964	0.0924	0.0885	0.0806	0.0727	0.0648	0.0569
0.50	0.0536	0.0527	0.0518	0.0501	0.0483	0.0465	0.0430	0.0395	0.0359	0.0324
1.00	0.0416	0.0410	0.0404	0.0391	0.0379	0.0366	0·0341	0.0316	0.0291	0.0266
1·20	0.0391	0.0385	0.0381	0.0367	0.0356	0.0343	0.0319	0.0296	0.0274	0.0250
1·40	0.0371	0.0365	0.0363	0.0347	0.0338	0.0324	0.0304	0.0283	0.0262	0.0239
1.60	0.0356	0.0348	0.0350	0.0332	0.0323	0.0312	0.0292	0.0272	0.0252	0.0232
1.80	0.0342	0.0336	0 0340	0.0321	0.0313	0.0302	0.0284	0.0265	0.0246	0.0226
2.00	0.0331	0.0327	0.0322	0.0313	0.0305	0.0296	0.0278	0.0260	0.0243	0.0225
2.50	0.0310	0.0306	0.0300	0.0294	0.0286	0.0278	0.0262	0.0246	0.0230	0.0214
3 00	0.0293	0.0290	0.0286	0.0279	0.0272	0.0265	0.0250	0.0236	0.0221	0.0207
3.50	0.0281	0.0278	0.0274	0.0268	0.0260	0.0254	0.0242	0.0228	0.0214	0.0201
4.00	0.0271	0.0268	0.0265	0.0259	0.0252	0.0246	0.0234	0.0221	0.0209	0.0196
4.50	0.0263	0.0260	0.0256	0.0251	0.0244	0.0238	0.0227	0.0216	0.0204	0.0192
5.00	0.0256	0.0253	0.0250	0.0244	0.0238	0.0233	0.0222	0.0211	0.0200	0.0189
5.50	0.0250	0.0246	0.0244	0.0238	0.0234	0.0228	0.0218	0.0207	0.0196	0.0186
6.00	0.0244	0.0242	0.0239	0 0234	0.0229	0.0224	0.0214	0.0204	0.0193	0.0183
6.50	0.0240	0.0236	0.0235	0.0230	0.0225	0.0220	0.0210	0.0200	0.0190	0.0181
7.00	0.0236	0.0233	0.0230	0.0226	0.0222	0.0217	0.0207	0.0198	0.0188	0.0179
7.50	0.0232	0.0229	0.0227	0.0222	0.0218	0.0214	0.0204	0.0195	0.0186	0.0177
8.00	0.0229	0.0226	0.0224	0.0220	0.0215	0.0211	0.0202	0.0193	0.0184	0.0175
8.50	0.0225	0.0223	0.0221	0.0216	0.0212	0.0208	0.0200	0.0191	0.0182	0.0174
9.00	0.0222	0.0220	0.0218	0.0214	0.0210	0.0206	0.0198	0.0189	0.0181	0.0173
9.50	0.0220	0.0218	0.0216	0.0212	0.0208	0.0204	0.0196	0.0188	0.0180	0.0172
10	0.0218	0.0216	0.0214	0.0210	0.0206	0.0202	0.0194	0.0186	0.0178	0.0170
11	0.0214	0 0212	0.0210	0.0206	0.0202	0.0198	0.0191	0.0184	0.0176	0.0168
12	0.0210	0 0208	0.0206	0.0203	0.0199	0.0195	0.0188	0.0181	0.0174	0.0166
13	0.0206	0.0205	0.0203	0.0199	0.0196	0.0193	0.0186	0.0179	0.0172	0.0165
14	0.0204	0.0202	0.0200	0.0197	0.0194	0.0190	0.0184	0.0177	0.0170	0.0164
15	0.0201	0.0200	0.0198	0.0195	0.0191	0.0188	0.0182	0.0175	0.0168	0.0162
16	0.0199	0.0197	0.0195	0.0192	0.0189	0.0186	0.0180	0.0174	0.0167	0.0161
17	0.0196	0.0195	0.0194	0.0190	0.0187	0.0184	0.0178	0.0172	0.0166	0.0160
18	0.0194	0.0193	0.0192	0.0188	0.0186	0.0183	0.0177	0.0171	0.0165	0.0159
19	0.0193	0.0191	0.0190	0.0187	0.0184	0.0181	0.0175	0.0169	0.0164	0.0158
20	0.0191	0.0189	0.0188	0.0185	0.0182	0.0180	0.0174	0.0168	0.0163	0.0157
25	0.0184	0.0183	0.0182	0.0179	0.0177	0.0174	0.0169	0.0164	0.0159	0.0154
30	0.0179	0.0178	0.0177	0.0174	0.0172	0.0170	0.0165	0.0161	0.0156	0.0152
35	0.0175	0.0174	0.0173	0.0171	0.0169	0.0167	0.0162	0.0158	0.0154	0.0150
40	0.0172	0.0171	0.0170	0.0168	0.0166	0.0164	0.0160	0.0156	0.0152	0.0148
50	0.0167	0.0166	0.0165	0.0163	0.0162	0.0160	0.0156	0.0153	0.0149	0.0146

The formula for a co-efficient of friction ζ for the two sheet-iron pipes, coated with bitumen, 7.72 and 11.22 inches in diameter, that has previously been mentioned, and which was constructed in a manner

Digitized by Google

similar to the one that has just been described, is constant for both diameters, and is dependent only upon the square root of the velocity ($\sqrt{v}$), viz.:

$$\zeta = 0.0126 + \frac{0.0105}{\sqrt{v}}.$$

This formula is platted for comparison with the experimental results from which it was determined (which are numbered from 338 to 348, and from 370 to 376 in Table No. 1), upon Diagrams Nos. 23 and 24.

FORMULAS FOR THE FLOW OF WATER IN PIPES HAVING INTERIOR SIDES SIMILAR TO NEW CAST-IRON PIPES.

An examination of Table No. 1 led the writer to the conclusion that there were one hundred and eighty-eight experiments which it would be advisable to utilize in constructing a formula for the flow of water in pipes having interior sides similar to new cast-iron pipes, that had been made with twenty-eight different pipes from 0.48 to 90 inches in diameter.

It was very difficult in a number of instances to determine the number of years of service that should limit what would generally be called a new or clean pipe, especially if the experiments under consideration had been made with pipes that were not originally coated with coal tar, or some preparation of a similar nature, unless there was positive evidence of the exact condition of their interior sides at the time that the experiments were made, as it has been demonstrated by chemical analysis that the interior sides of cast-iron pipe oxidize much faster with slightly alkaline and aerated waters flowing through them than they do with other waters differently constituted. In the absence of more reliable data, the writer has generally been governed in cases of this kind by the values of the co-efficients of friction (ζ).

After carefully studying and platting the experimental results in a manner similar to that mentioned in the description of the construction of "A new formula for the flow of water in pipes having very smooth interior sides," the writer made up his mind that he could not construct a formula that would compare as satisfactorily with the experimental results, as two formulas by Henry Darcy, which are to be found on pages 254, 258 and 368 of his work entitled "Recherches expérimentales relatives au mouvement de l'eau dans les tuyaux."

Digitized by Google

The formulas for obtaining the co-efficients of friction (ζ), which is a factor of the equation $h_f = \zeta \frac{l}{d} \frac{v^2}{2g}$ as worked out from these formulas (Darcy's) by the writer, and reduced to English measures are as follows:

$$\zeta = 0.0198920 + \frac{0.00166573}{d}.$$

$$\zeta = 0.017379 + \frac{0.0015965}{d} + \frac{0.0040723 + \frac{0.000020816}{d^2}}{v}.$$

Darcy recommends the formula from which the first formula for the co-efficient of friction ζ was deduced, for general use, and it is ordinarily known as "Darcy's formula." As may be seen in the first formula, the value of the co-efficient ζ, is entirely dependent upon the diameter of the pipe. Darcy did not fail to recognize, however, that the velocity also entered into the problem, but considered (after analyzing the experimental results from which the formula was derived), that when the velocity is more than 0.33 feet per second, especially if the pipes have been in use a short time, the term relative to it, which would enter into the formula for the co-efficient of friction ζ, could be entirely eliminated, without the reliability of the formula being appreciably affected. The formula, from which the second formula for a co-efficient of friction ζ was deduced, was intended by Darcy to be used for all velocities, and to apply more especially to new pipes. (The same experimental results were used in constructing both of these formulas.)

The experimental co-efficients of friction (ζ) that were worked out from the one hundred and eighty-eight experiments, and co-efficients of friction worked out from Darcy's formulas, and several other well-known formulas, are platted for comparison on diagrams from No. 15 to No. 22, and on No. 25.

The formulas for obtaining the co-efficients of friction from the formulas last mentioned, as arranged by the writer (the first three of which have previously been referred to), are as follows:

From the formula of Prony,

$$\zeta = 0.02733 + \frac{0.004464}{v}$$

From the formula of Eytelwein,

$$\zeta = 0.0220 + \frac{0.005754}{v}.$$

Digitized by Google

From the formula of Weisbach,

$$\zeta = 0.01439 + \frac{0.01704}{\sqrt{v}}$$

From the formula of Kutter,

$$\zeta = \frac{2g}{\left(\frac{20.83 + \frac{0.9056}{n} + \frac{0.0014038}{J}}{1 + \left(41.66 + \frac{0.0028075}{J}\right)\frac{2n}{\sqrt{d}}}\right)^2}$$

For facility of investigation, when it could be done without affecting the results sought after before reaching the fourth decimal place, the greater part of the experimental results that were obtained with pipes having diameters very nearly alike were classified under a common nominal diameter, as will be seen upon the diagrams.

The nominal diameters of the pipes that were used in making the experiments, the results of which are platted on the diagrams, the numbers that the experiments are given in Table No. 1, and the numbers of the diagrams upon which the results are platted, are as follows:

0.48-inch, Tabular numbers 15 to 27. Diagram No. 25.
1.047-inch, Tabular numbers 61 to 77. Diagram No. 25.
1.55-inch, Tabular numbers 212 to 223. Diagram No. 25.
3.22-inch, Tabular numbers 283 to 295. Diagram No. 15.
5.39-inch, Tabular numbers 309 to 328. Diagram No. 16.
7.40-inch, Tabular numbers 329 to 337. Diagram No. 17.
11.69-inch, Tabular numbers 365 to 369, 377 to 396, 405 to 408. Diagram No. 18.
15.00-inch, Tabular numbers 409 to 422. Diagram No. 19.
19.69-inch, Tabular numbers 423 to 456. Diagram No. 20.
30.00-inch, Tabular numbers 467 to 485. Diagram No. 21.
48.00-inch, Tabular numbers 513 to 517. Diagram No. 21.
90.00-inch, Tabular numbers 518 to 520. Diagram No. 22.

There are platted upon diagrams Nos. 18, 19, 20 and 21, a number of experimental results that should probably not be given as much weight in making a comparison, as the other experimental results which are platted upon the same diagrams, viz.: Upon diagram No. 18, the results that were obtained from ten of the experiments made by Simpson, on account of old pipe having been used; upon diagrams Nos. 19 and 20, the results obtained from three of the experiments made by the Edinburgh Water Company, and five of the experiments made by Simpson, for the same reason; and upon diagram No. 21, the results obtained

Digitized by Google

from eighteen experiments made by Darrach with pipes 30 inches in diameter, on account of allowances that had to be made, in computation, for the resistance of check-valves which were located upon the pipes.

As will be seen by an examination of the diagrams, the co-efficients of friction (ζ) worked out from Darcy's formulas agree very well in the majority of cases with the experimental results, and much better than any of the other formulas that are platted. Therefore, in the opinion of the writer, the formulas of Darcy are safe and reliable to use in general practice. The former, which is represented by a straight line on the diagrams, for velocities including and exceeding 0.33 feet per second, and the latter, which is represented by a curved line on the diagrams, for velocities of less than 0.33 feet per second. The former is also to be recommended for its simplicity, as well as for its accuracy, and also for the convenience with which it can be used in both the simple and extended forms of equations previously mentioned and recommended by the writer, for the flow of water in pipes.

Of the other co-efficients of friction (ζ), that are platted on the diagrams, which have been worked out from the formulas of different authorities, the writer will only mention in detail those that have been derived from the well-known and very valuable formula of Kutter, which during the last few years has been so much discussed and extensively used for computing the flow of water through masonry conduits. There seems to be a general impression among a number of engineers that this formula can be applied to iron pipes, although Kutter does not give any co-efficients of roughness, in connection with it, which were obtained from experiments that were made with iron pipes. In order to ascertain if this impression was correct, the writer used in computing the results for comparison for this formula, three co-efficients of roughness as given by Kutter, which in his opinion would be the most likely to apply to iron pipes, viz.: 0.010 which is for a surface of plaster in pure cement, 0.011 which is for a surface of plaster in cement with one-third sand, and 0.013 which is for a surface of brickwork or ashlar. The results obtained by using these three co-efficients of roughness are platted on the diagrams from No. 16 to No. 22, and in the writer's opinion neither of them compare as favorably with the experimental results as the formulas of Darcy; also the same co-efficient of roughness seems not to apply to all sizes of pipe, for instance, 0.010 applies best to pipes having nomi-

Digitized by Google

nal diameters of 5.39 and 7.40 inches, 0.011 to pipes having nominal diameters of 11.69, 16 and 19.69 inches, and 0.013 to pipes having diameters of 30 and 48 inches.

The results obtained from the experiments made by Clarke with a brick tunnel 90 inches in diameter are simply platted on Diagram No. 22, in order to show to what an extreme range of diameters the formulas of Darcy can be safely applied.

OLD CAST-IRON PIPES LINED WITH DEPOSIT, AND THE SAME CLEANED.

It was found by examining Table No. 1 that there were fifty-eight experiments that had been made with eight different old cast-iron pipes lined with deposit, from 1.43 to 36 inches in diameter, and twenty-two experiments that had been made with three of these pipes after they had been cleaned.

A preliminary study of the results obtained from the experiments that were made with the old cast-iron pipes lined with deposit convinced the writer that he could not satisfactorily construct a general formula from the data at hand. He was able, however, to construct three individual formulas for co-efficients of friction (ζ). In one instance with two sets of results obtained from experiments made with two different pipes of the same diameter, and in two instances with three sets of results obtained from experiments made with three different pipes of the same diameter, which are as follows:

For 20-inch pipe, $\zeta = 0.0156 + \frac{0.116}{v}$.

For 36-inch pipe, $\zeta = \frac{0.159}{v} - 0.0143$.

For 36-inch pipe, $\zeta = \frac{0.1835}{v} - 0.0225$.

(The same experimental results were used in constructing the two latter formulas.)

All of the co-efficients of friction (ζ) that have been derived from experiments that were made with old pipes lined with deposit (with the exception of ten that were obtained from experiments made with one pipe 30 inches in diameter), are platted upon diagrams Nos. 26 and 28, for comparison with these formulas, and with one worked out by the writer from a formula by Darcy for the flow of water in pipes slightly encrusted with deposit, viz.,

$$\zeta = 0.03978 + \frac{0.03332}{d}.$$

Digitized by Google

As the writer does not consider these formulas for old pipes lined with deposit of much value, they will not be further discussed.

The results that were obtained from experiments that were made with old cast-iron pipes after having been cleaned, are simply platted on diagram No. 27 for the purpose of comparing them with the formula for a co-efficient of friction ζ, worked out from a formula by Darcy for the flow of water in new cast-iron pipes, which has been previously mentioned, viz.,

$$\zeta = 0.019892 + \frac{0.00166573}{d}.$$

The diameters of the pipes that were used in making the experiments, the results of which have just been mentioned, the numbers that the experiments are given in Table No. 1, and the numbers of the diagrams upon which the results are platted, are as follows:

1.43-inch, Tabular numbers 161 to 174.	Diagrams Nos. 26 and 27.
3.15-inch, Tabular numbers 270 to 282.	Diagrams Nos. 26 and 27.
9.63-inch, Tabular numbers 349 to 364.	Diagrams Nos. 26 and 27.
20.00-inch, Tabular numbers 457 to 466.	Diagram No. 26.
30.00-inch, Tabular numbers 486 to 495.	(Not platted:)
36.00-inch, Tabular numbers 496 to 512.	Diagram No. 28.

THE CO-EFFICIENTS OF RESISTANCE OF THE FLOW OF WATER IN PIPES FOR ENLARGEMENTS, CONTRACTIONS, ELBOWS AND CURVES.

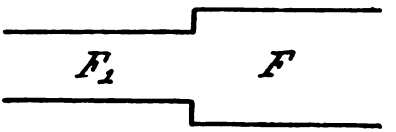
Should the forms of formulas that have been recommended by the writer for the flow of water in pipes, be used and extended to cover conditions other than those relating to straight pipes, one or more of the four co-efficients of resistance, for enlargements, contractions, elbows and curves, will probably be required, in order to do so. Therefore, as the writer has not anything original to present pertaining to these co-efficients, he has taken the following data relating to them, nearly entire, from the valuable work of Professor Weisbach, entitled, "A Manual of the Mechanics of Engineering and of the Construction of Machines," which was translated from the fourth German Edition, by E. B. Coxe, A. M., in 1872.

Sudden Enlargement.

$$\mu = \left(\frac{F}{F_1} - 1\right)^2, \quad h_f = \mu \frac{v^2}{2g}.$$

Digitized by Google

Fig. 3.



The following table is calculated according to the formula.

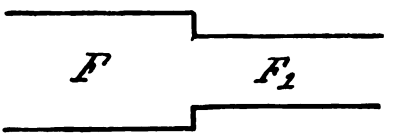
$\frac{F}{F_1}$	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9
μ	.01	.04	.09	.16	.25	.36	.49	.64	.81

$\frac{F}{F_1}$	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0
μ	1.00	2.25	4.00	6.25	9.00	16.00	25.00	36.00	49.00

CONTRACTION.

$$\phi = \left(\frac{1 - 0.64}{0.64}\right)^2 = 0.316, \; h_f = \phi \frac{v^2}{2g}.$$

Fig. 4.



but ϕ is increased by the resistance at the entrance into the pipe and by the friction of the water in the interior portion of the tube, to 0.505.

ELBOWS.

$$\tau = 0.9457 \sin \frac{2\rho}{2} + 2.047 \sin \frac{4\rho}{2}, \; h_f = \tau \frac{v^2}{2g}.$$

Fig. 5.

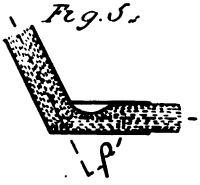


Fig. 6.

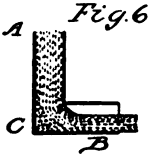


Fig. 7.

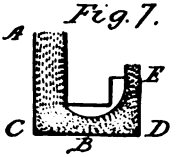
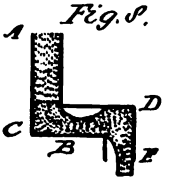


Fig. 8.



Digitized by Google

The following table contains a series of co-efficients of resistance calculated, according to the formula, for different angles of deviation :

$\frac{\rho}{2}$	10°	20°	30°	40°	45°	50°	55°	60°	65°	70°
τ	.046	.139	.364	.740	.984	1.260	1.556	1.861	2.158	2.431

If to one elbow *ACB*, Fig. 6, another elbow is joined, as is shown in Figs. 7 and 8, a peculiar but at the same time easily explicable phenomenon of efflux is observed. The elbow *BDE*, Fig. 7, which turns the stream to the same side as the elbow *ACB*, Fig. 6, produces no further contraction of the stream, and, therefore, for efflux with full cross section, τ is no greater than for a simple elbow *ACB*. But if the other elbow *BDE*, Fig. 8, turns the stream to the opposite side, the contraction is a double one, and the co-efficient of resistance is consequently twice as great as for a single elbow. If, finally, *BDE* is so joined to *ACB* that *DE* stands at right angles to the plane *ABD*, τ then becomes about one and one-half times as great as for the single elbow *ACB*.

Curved Pipes.

For curved pipes with circular cross-sections:

$$\psi = 0.131 + 1.847 \left(\frac{d}{2r}\right)^{\frac{7}{2}}, \quad h_f = \psi \frac{v^2}{2g}.$$

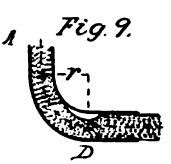


Fig. 9.

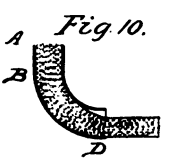


Fig. 10.

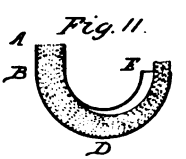


Fig. 11.

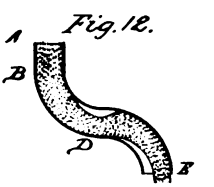


Fig. 12.

The following table is calculated according to the formula:

$\frac{d}{2r}$	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
ψ	.131	.138	.158	.206	.294	.440	.661	.977	1.408	1.978

Digitized by Google

If one curve *BD*, Fig. 10, is terminated by another, which turns the stream further in the same direction, if *eg*, the axis of the pipe forms a semicircle, like *BDE*, Fig. 11, the contraction is not changed and ψ has the same value as for the pipe in Fig. 10, which forms a quadrant. If, on the contrary, a curve *DE*, Fig. 12, which turns the stream in the opposite direction, is attached to the first one, an eddy is formed between the two, and a second contraction of the stream takes place by which the resistance (ψ) is nearly doubled.

Digitized by Google

PLATE I
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

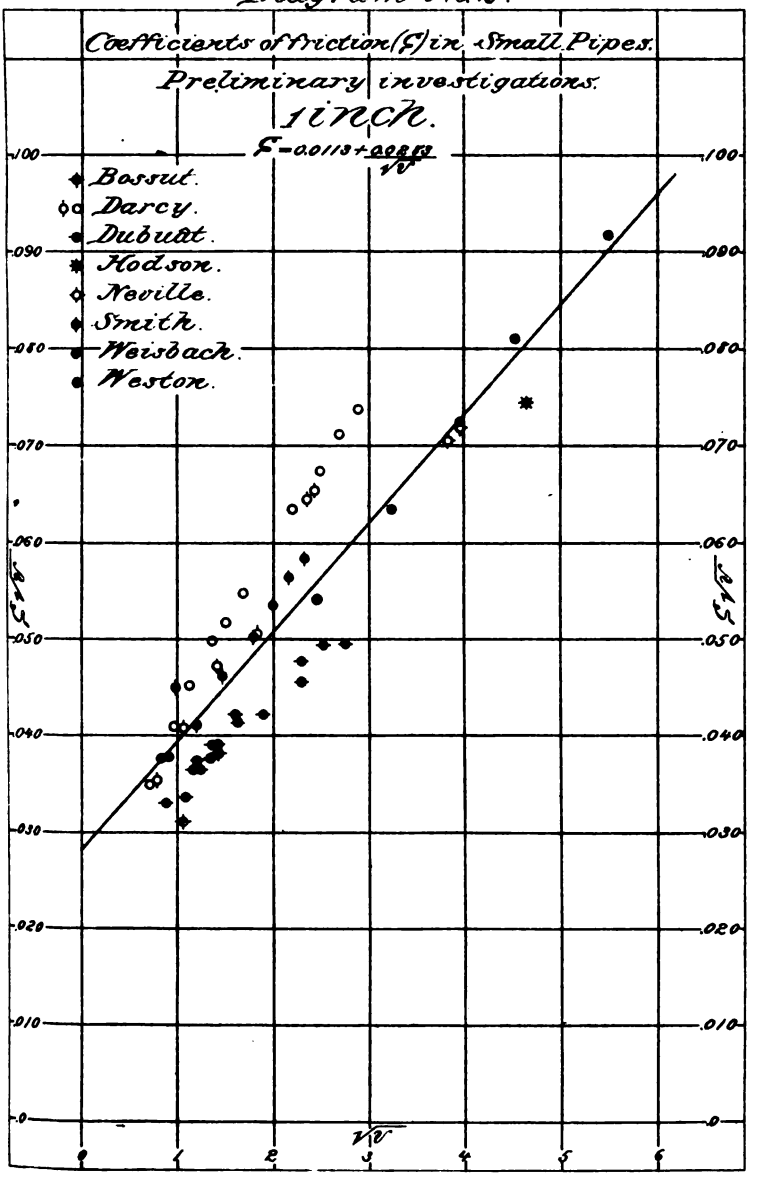
WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 1.

Digitized by Google

Digitized by Google

Diagram No 2.



Digitized by Google

Digitized by Google

PLATE III.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 3.

Digitized by Google

Digitized by Google

PLATE IV
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 4.

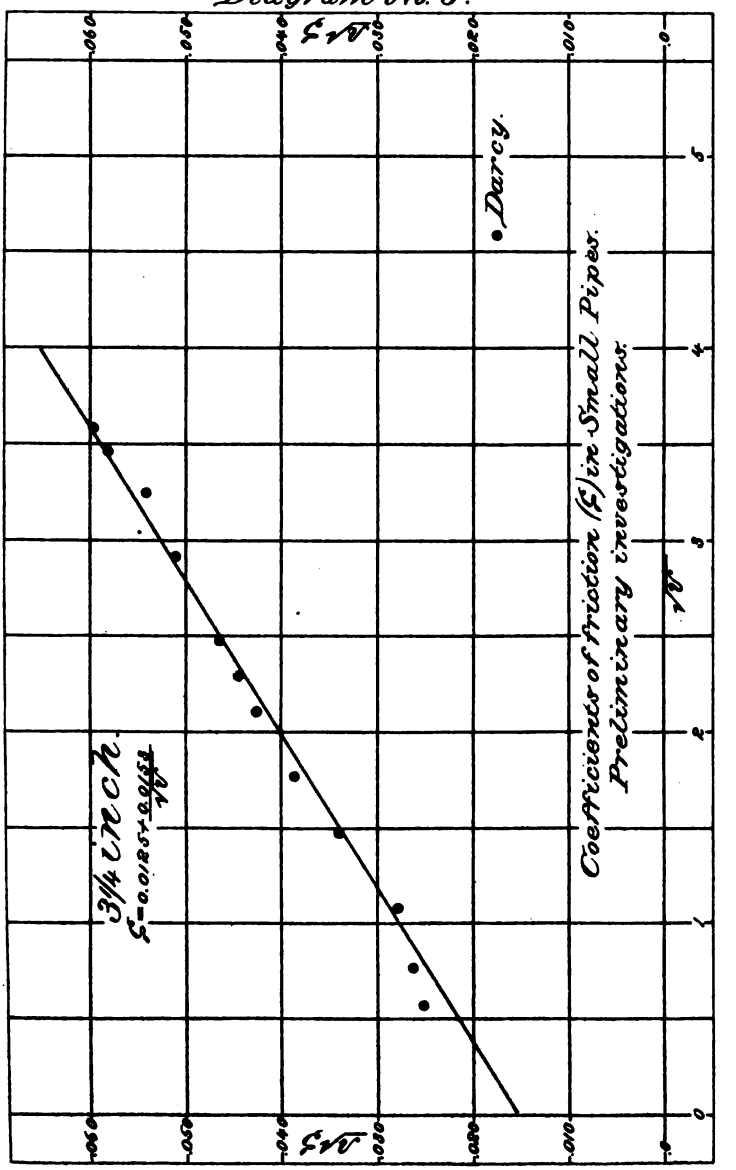
Digitized by Google

Digitized by Google

PLATE V.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON.
ON
FLOW OF WATER IN PIPES.

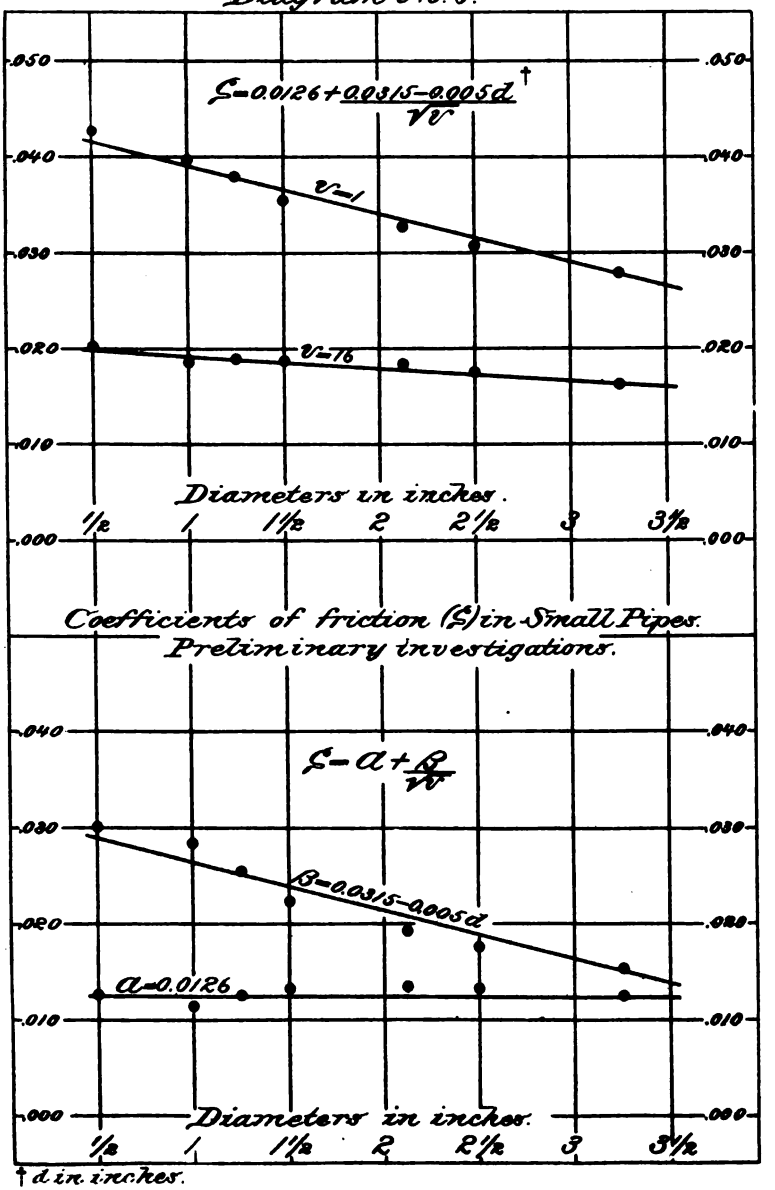
Diagram No. 5.



Digitized by Google

Digitized by Google

Diagram No. 6.



† d in inches.

Digitized by Google

Digitized by Google

PLATE VII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431.

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram Nº 7.

Coefficients of Friction (f) in Small Pipes.

Comparison of formulae with experiments.

1/2 INCH.

$f = 0.0126 + 0.0315 \div \sqrt{v}$ †

Experiments.

- ● Darcy 0.551 inch.
- ◆ Smith 0.622 "
- ◆ Weisbach 0.622 "
- ✱ Weisbach 0.805 "
- ✲ Reynolds 0.500 "

Formulae.

- Weston ——
- Darcy - - - -
- Weisbach ----------
- Fryelwerio — — —
- Prony ——

Velocity.

† d in inches.

Digitized by Google

Digitized by Google

PLATE VIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 8.

Coefficients of friction (f) in Small Pipes.
Comparison of formulæ with experiments.
3/4 inch.
$f = .0076 + \frac{.0081}{\sqrt{v}}$

Experiments.
• Smith & Wisick.

Formulæ.
Weston
Darcy
Weisbach.
Eytelwein
Prony

Velocity.

† d in inches.

Digitized by Google

Digitized by Google

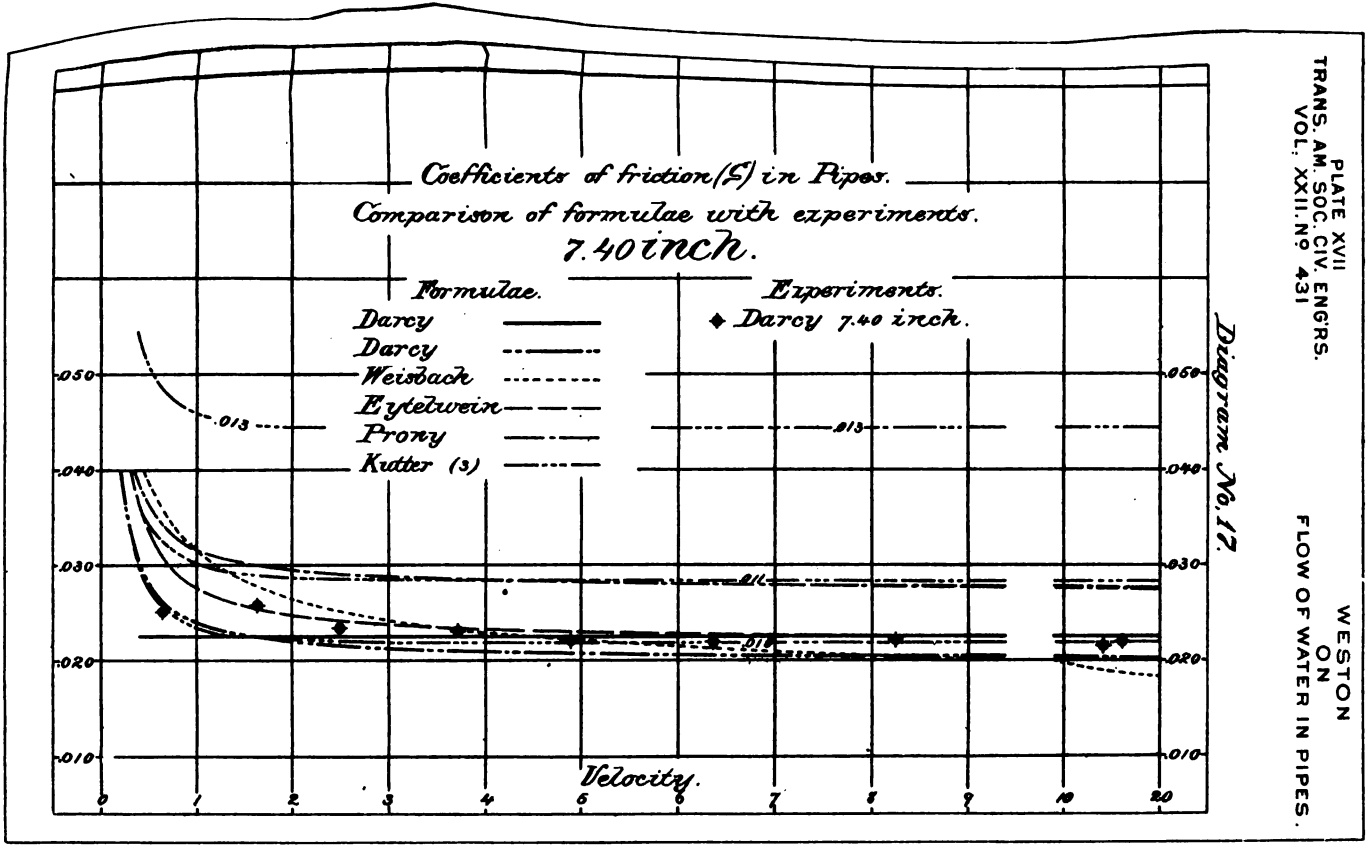


Diagram No. 17.

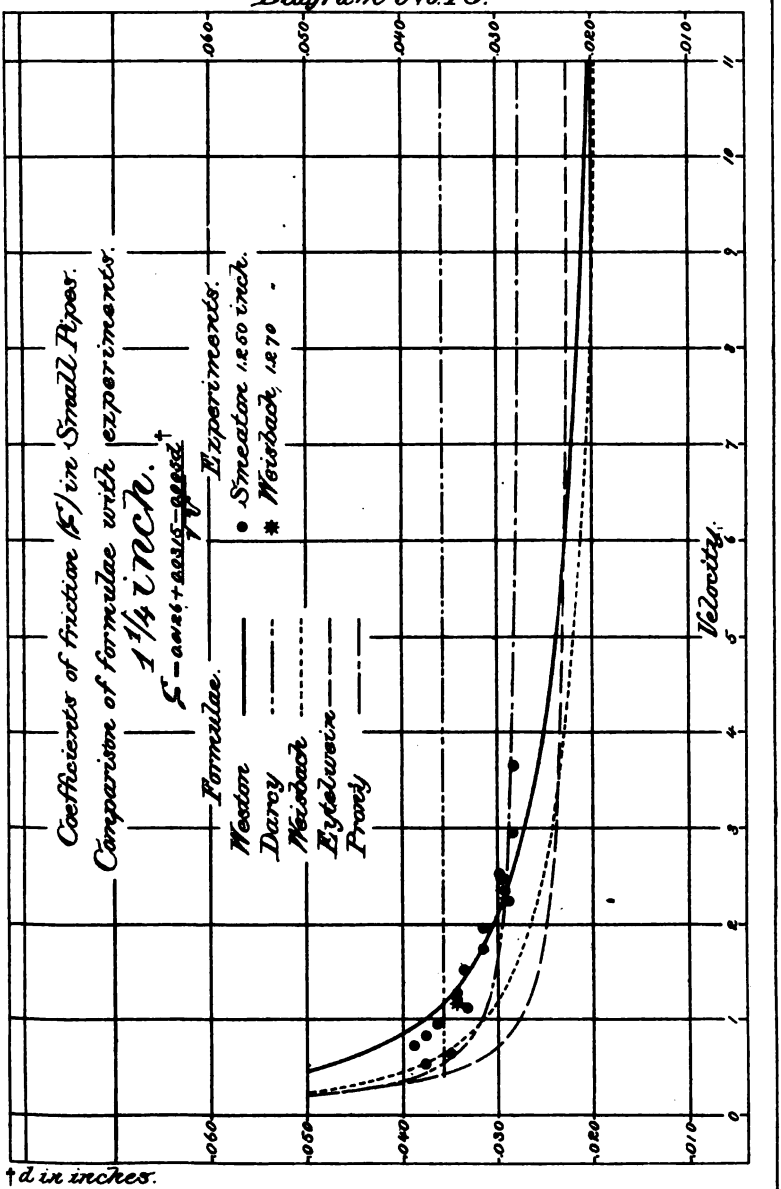
Digitized by Google

Digitized by Google

PLATE X
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

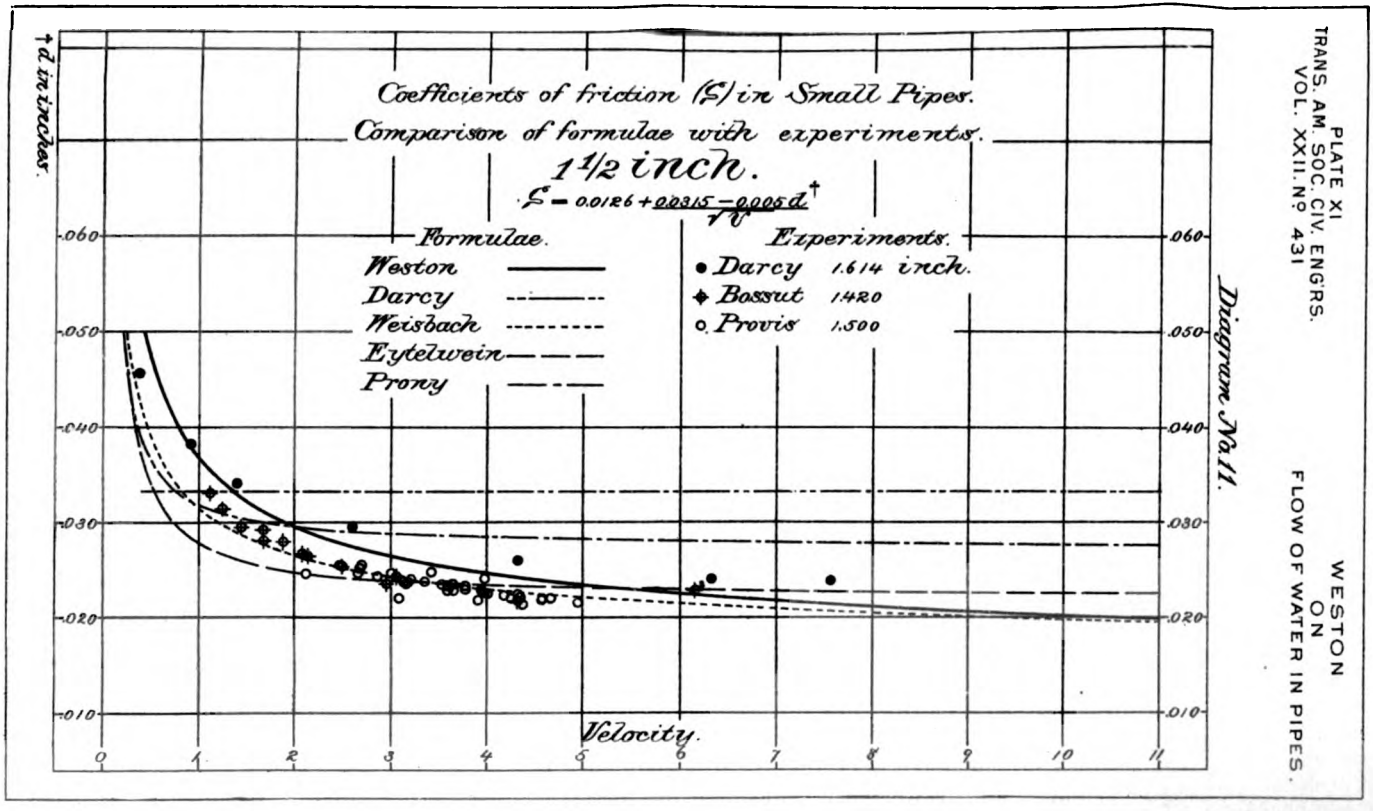
Diagram No. 10.



† d in inches.

Digitized by Google

Digitized by Google



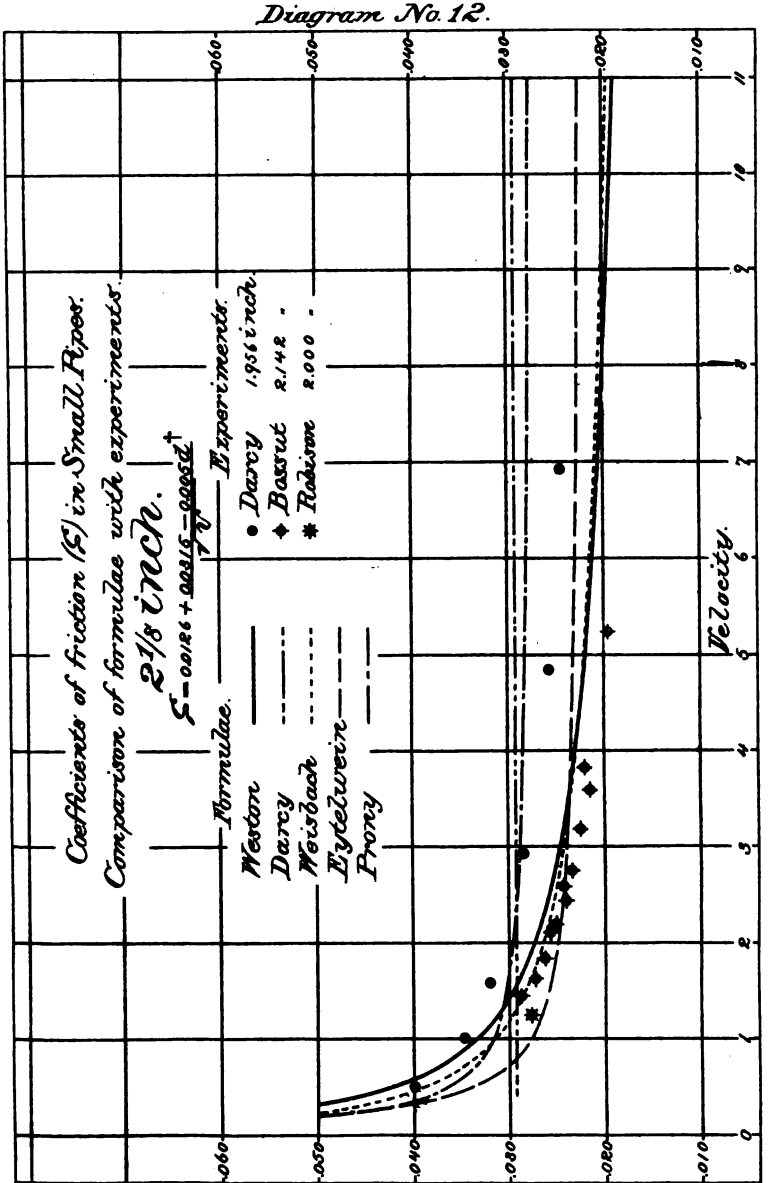
Digitized by Google

Digitized by Google

PLATE XII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431.

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 12.



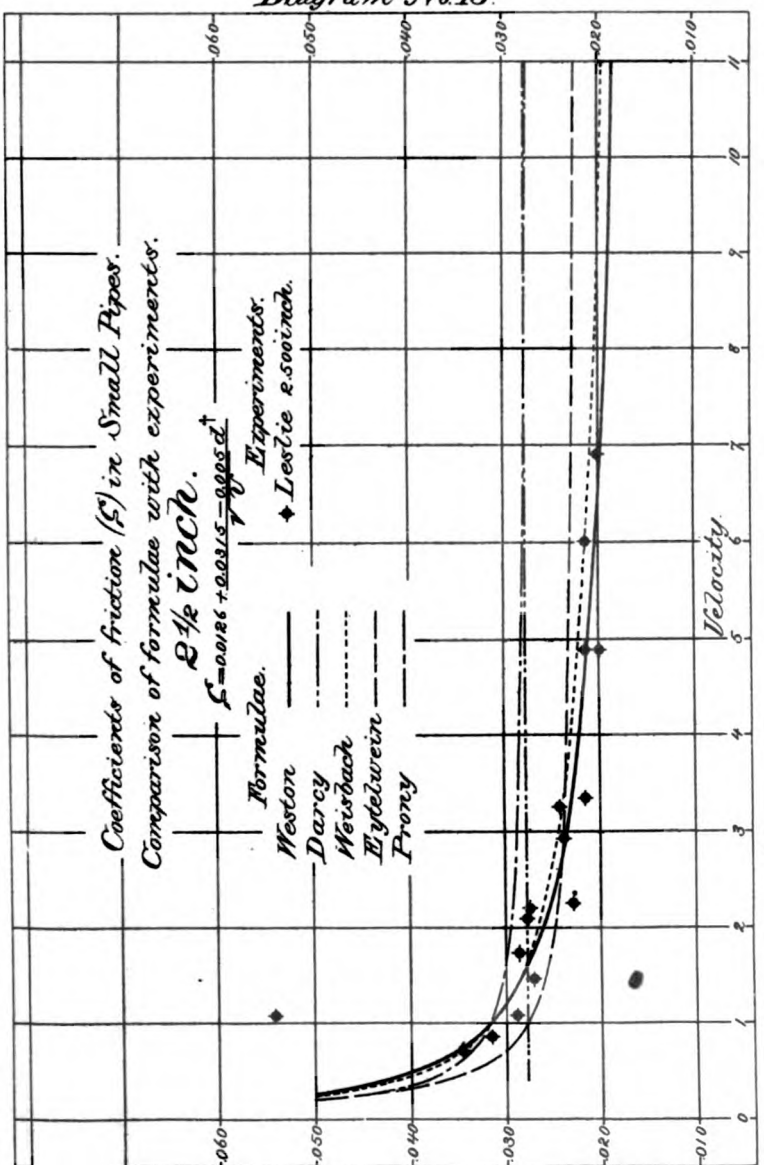
† d in inches.

Digitized by Google

PLATE XIII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 13.



† d in inches.

Digitized by Google

PLATE XIV
TRANS. AM. SOC. CIV ENG'RS.
VOL. XXII. Nº 431.

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 14.

Coefficients of friction (f) in Small Pipes.

Comparison of formulae with experiments.

3¼ inch.

f = area + basis ÷ velocity

Experiments.
• Darcy 3.25 inch.

Formulae.
Weston ——
Darcy - - - -
Weisbach ·····
Fteley & Stearns — — —
Prony — · — · —

.060 .050 .040 .030 .020 .010

0 1 2 3 4 5 6 7 8 9 10 11 12

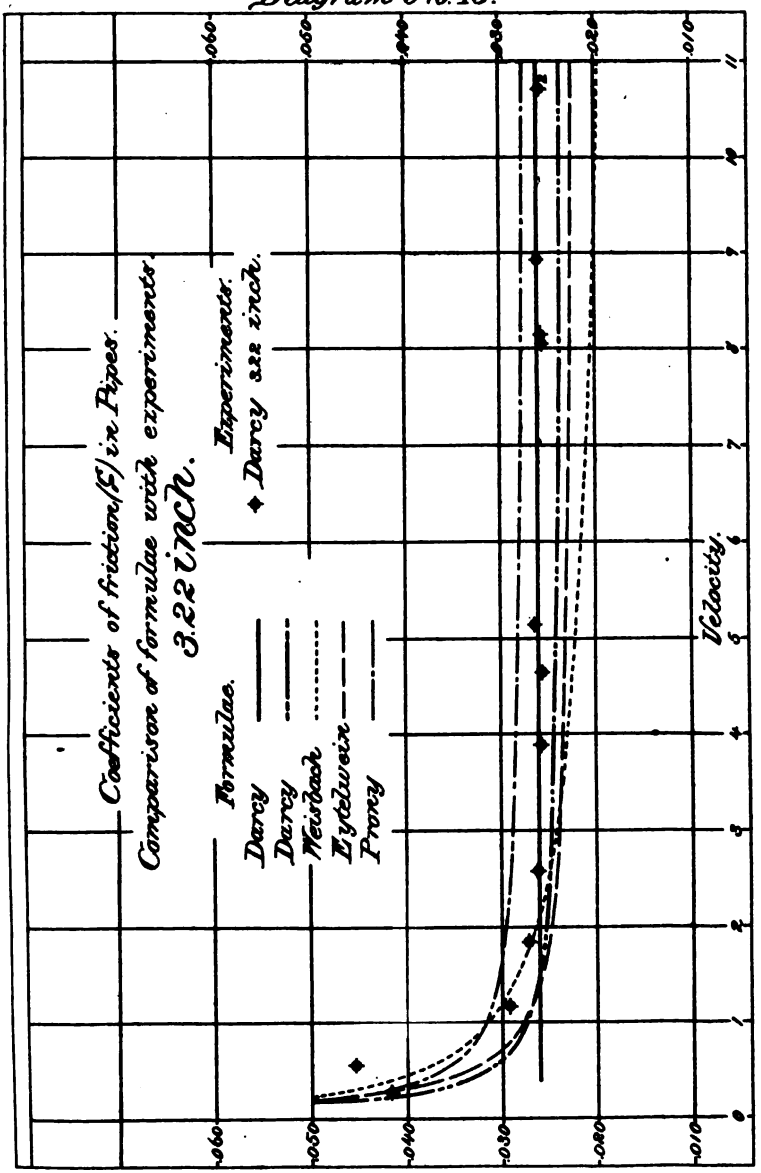
Velocity.

† din inches.

Digitized by Google

Digitized by Google

Diagram No. 15.



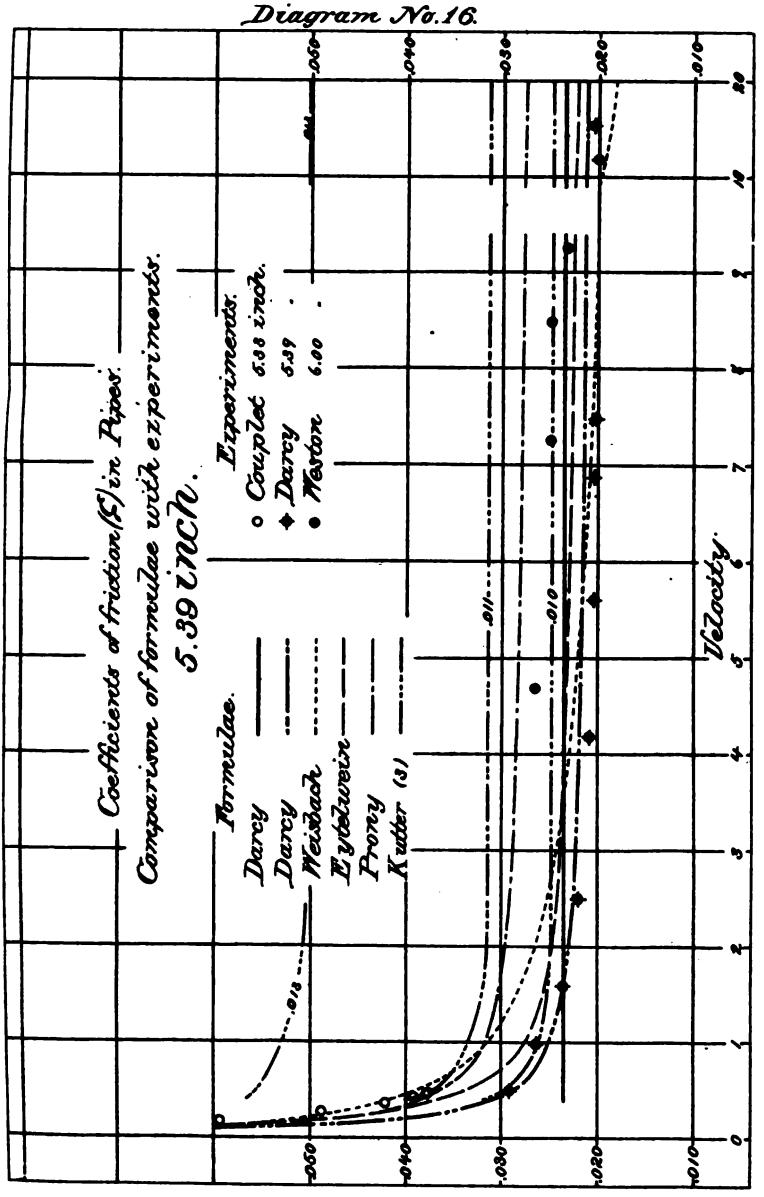
Digitized by Google

Digitized by Google

PLATE XVI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 16.



Digitized by Google

Digitized by Google

Diagram No. 17.

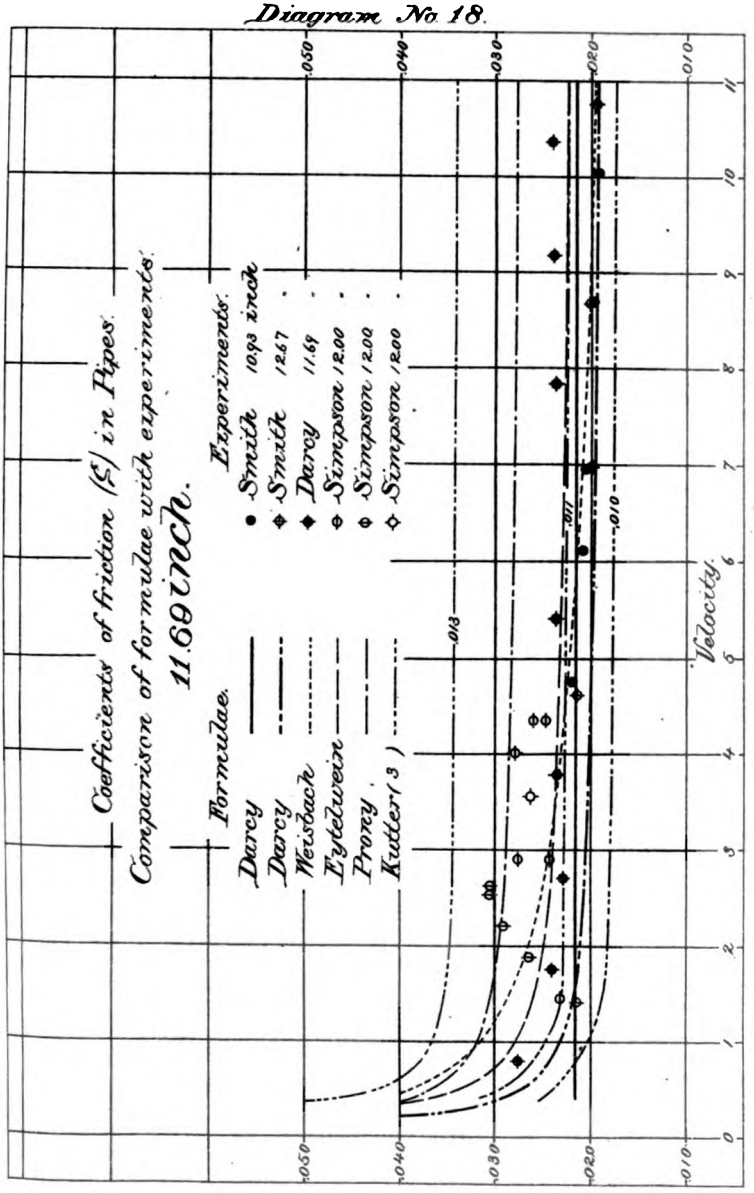
Digitized by Google

Digitized by Google

PLATE XVIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431.

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 18.



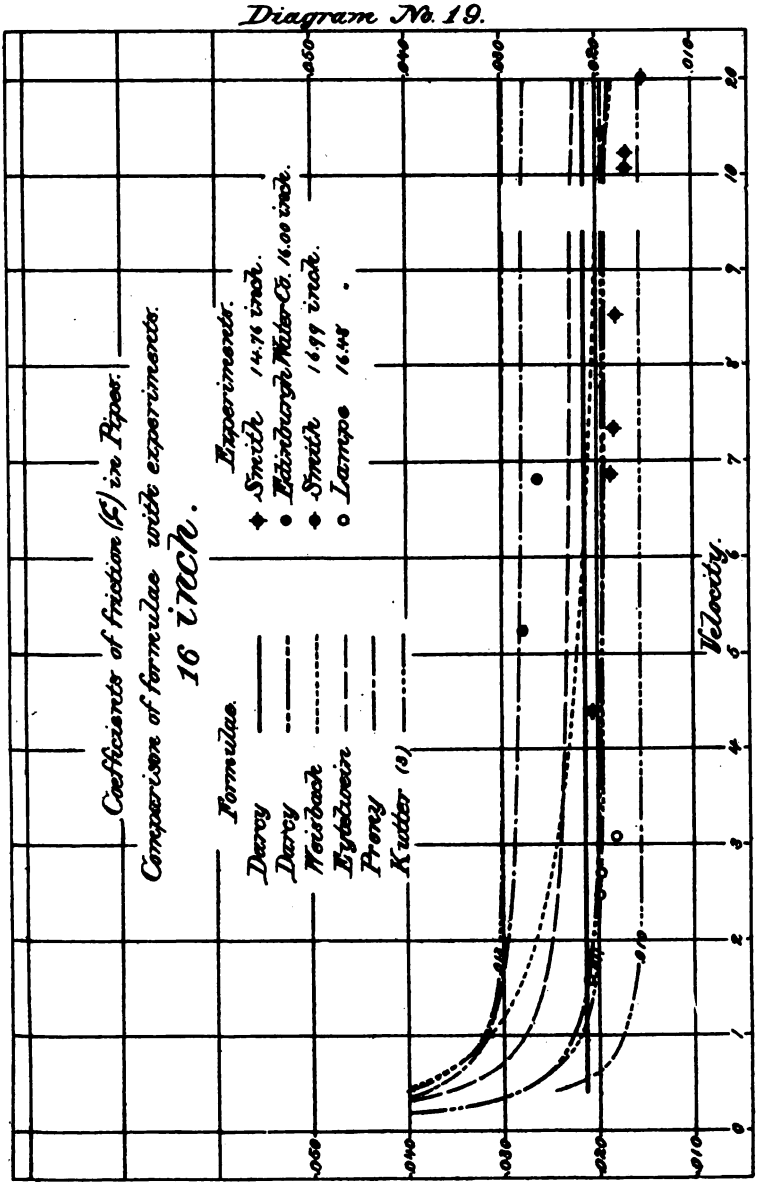
Digitized by Google

Digitized by Google

PLATE XIX
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. No 431.

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 19.



Digitized by Google

Digitized by Google

PLATE XX
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 20.

Coefficients of friction (f) in Pipes.

Comparison of formulae with experiments.

19.69 inch.

Formulae.

Darcy ——
Darcy - - -
Weisbach .....
Eytelwein — — —
Prony —·—·—
Kutter (S.) .....

Experiments.

◆ Darcy 19.69 inch.
◆ Fanning 2000 .
● Simpson 19.00 .
○ Brush 20.00 .

.050
.040
.030
.020
.010

.013
.011
.010

0 1 2 3 4 5 6 7 8 9 10 11

Velocity.

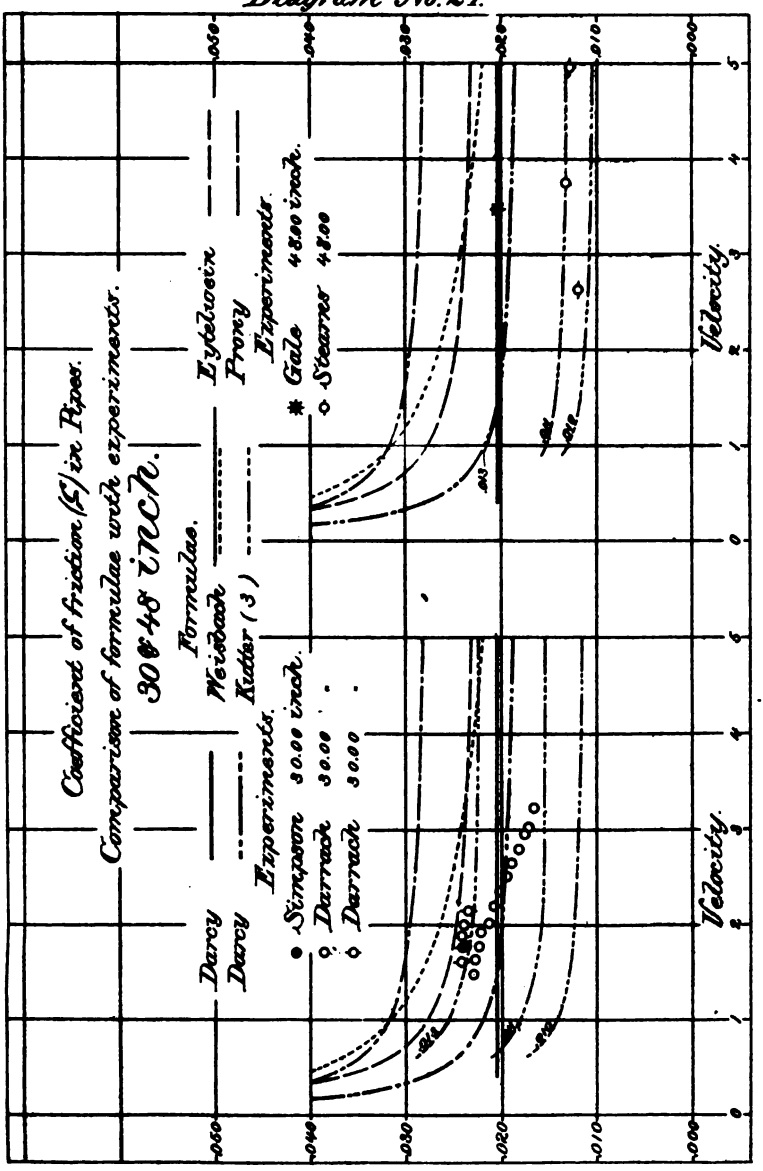
Digitized by Google

Digitized by Google

PLATE XXI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

WESTON
ON
FLOW OF WATER IN PIPES.

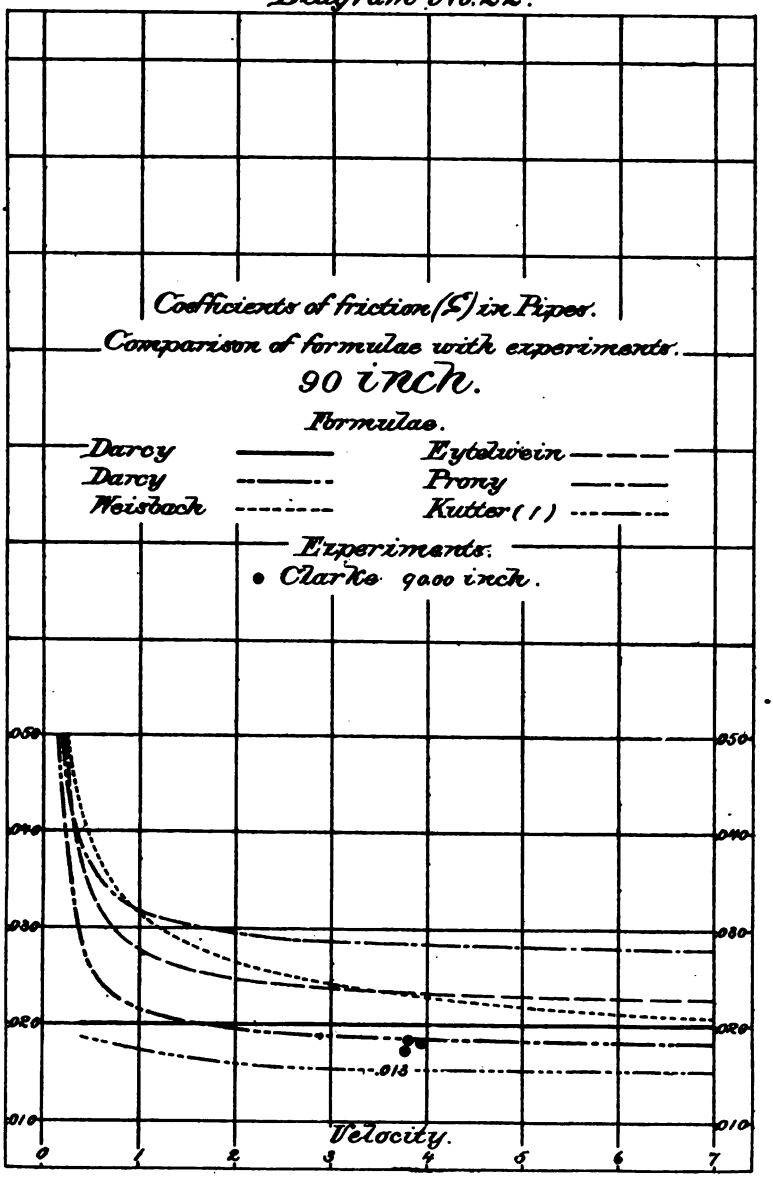
Diagram No. 21.



Digitized by Google

Digitized by Google

Diagram No. 22.



Digitized by Google

Digitized by Google

Diagram No. 23.

Digitized by Google

Digitized by Google

PLATE XXIV
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

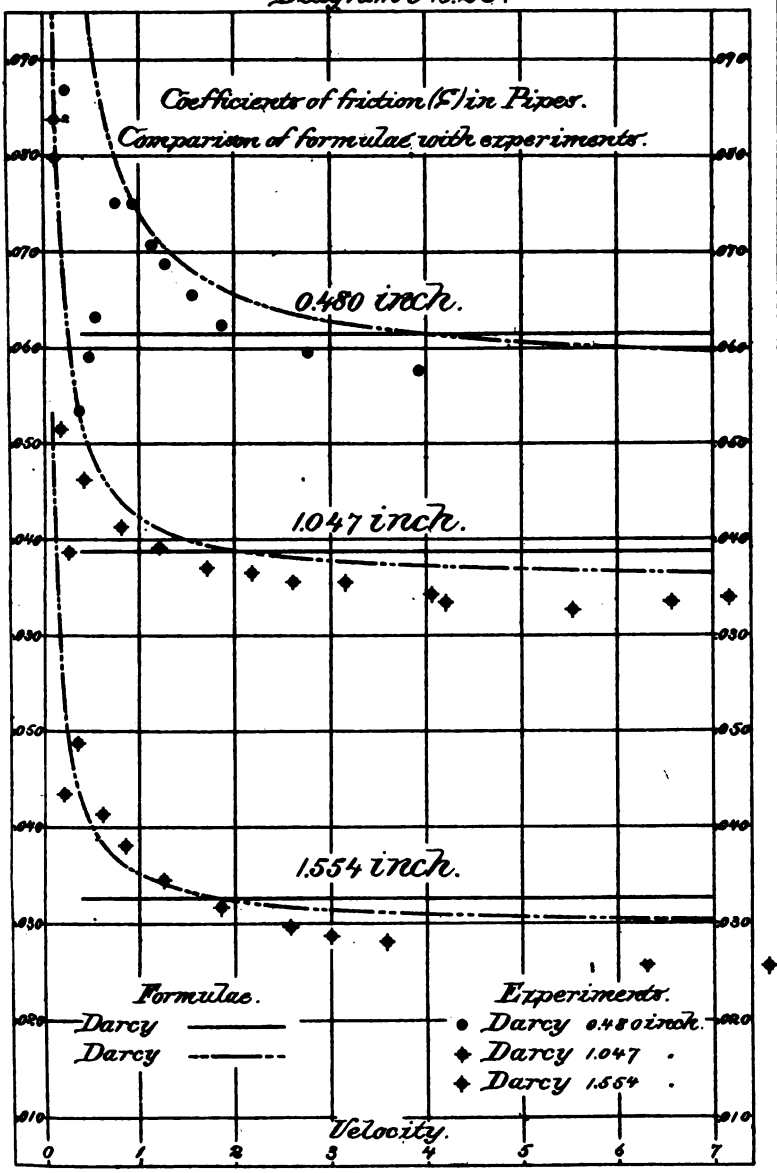
WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 24.

Digitized by Google

Digitized by Google

Diagram No. 25.



Digitized by Google

Digitized by Google

PLATE XXVI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 431

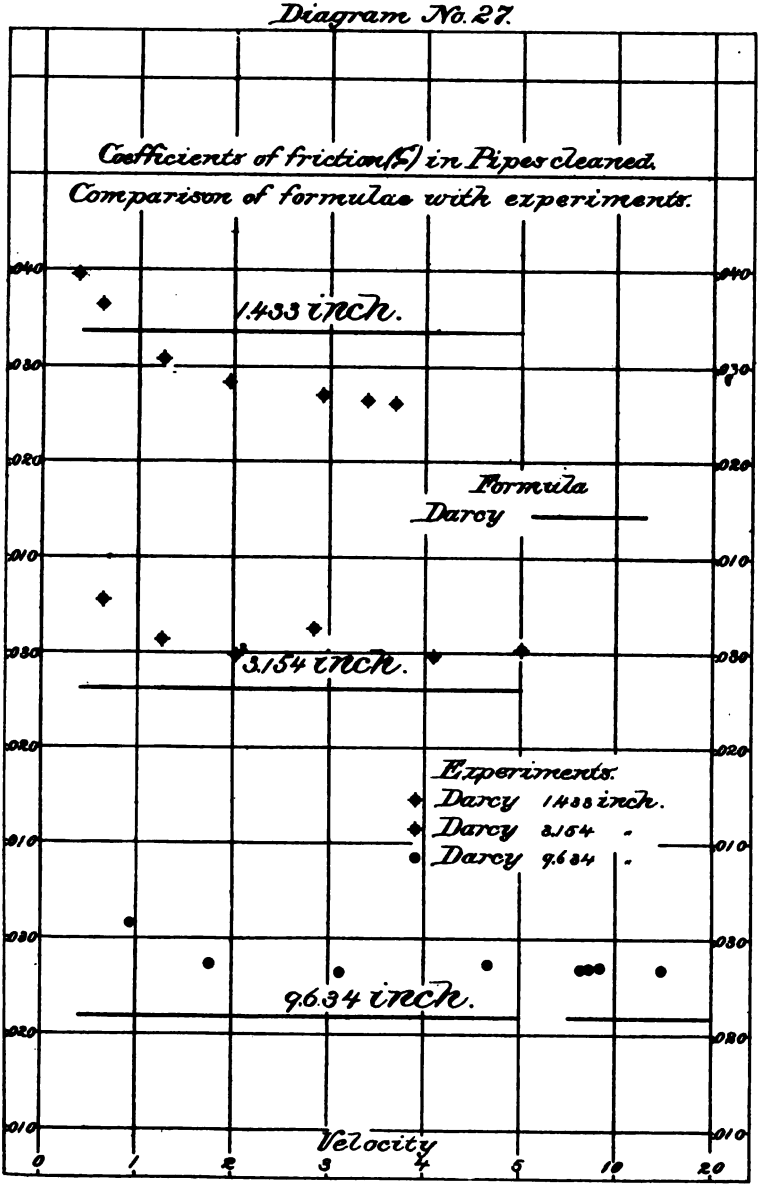
WESTON
ON
FLOW OF WATER IN PIPES.

Diagram No. 26.

Digitized by Google

Digitized by Google

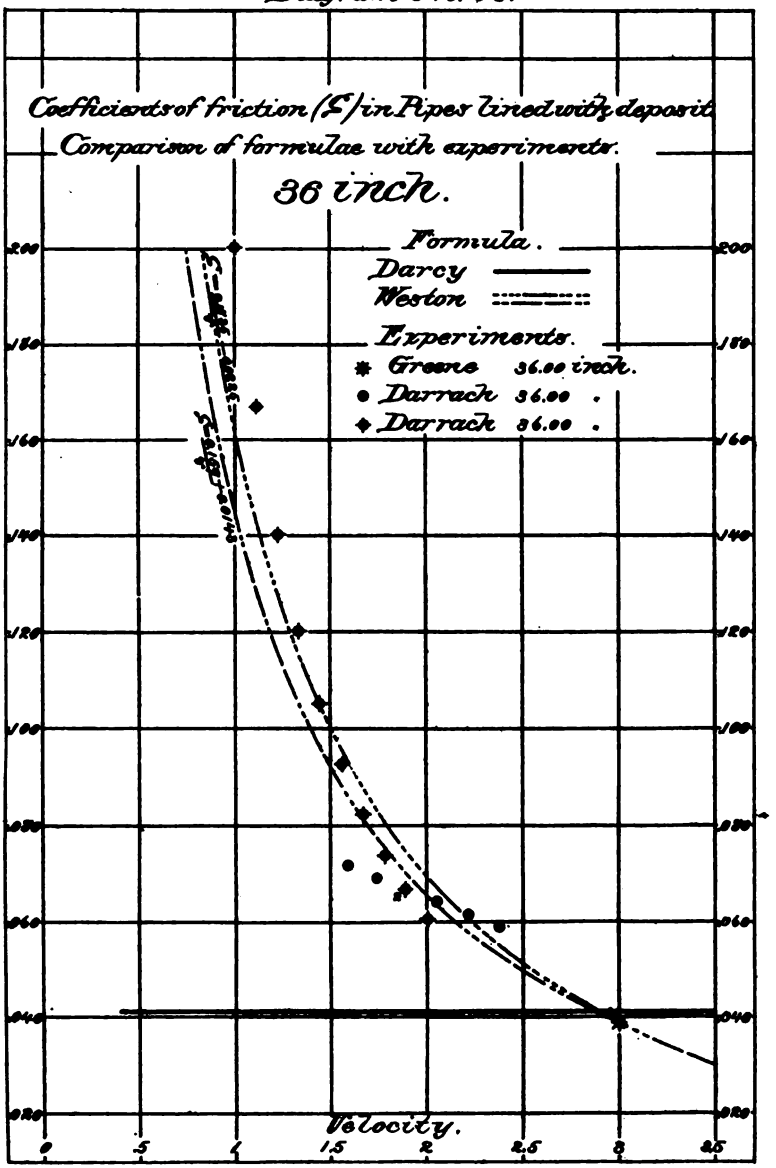
Diagram No. 27.



Digitized by Google

Digitized by Google

Diagram No. 28.



Digitized by Google

Digitized by Google

APPENDIX.

Loss of Head due to Friction of Water in Pipes having very Smooth Interior Sides similar to Brass and Lead Pipes, etc.

Table calculated from a formula by Edmund B. Weston, M. Am. Soc., C. E., for Pipes 100 Feet in Length, showing:

1st. The mean velocity of the water flowing in the pipe, in feet per second.

2d. The interior diameter of the pipe.

3d. The loss of head due to friction, in feet.

4th. The number of gallons of water delivered per minute.

Loss of Head due to Friction of Water flowing in Pipes 100 Feet in length, having very Smooth Interior Sides.

(*Weston's Formula.*)

Velocity in Feet per Second.	Internal Diameter of Pipe in Inches.									
	½		⅝		¾		1		1¼	
	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.
1	1.55	0.61	1.22	0.96	1.00	1.38	0.73	2.45	0.56	3.83
2	4.94	1.22	3.9	1.91	3.21	2.75	2.34	4.90	1.82	7.65
3	9.85	1.84	7.79	2.87	6.41	4.13	4.68	7.34	3.65	11.48
4	16.18	2.45	12.79	3.83	10.54	5.51	7.72	9.79	6.02	15.30
6	32.83	3.67	25.99	5.74	21.43	8.26	15.73	14.69	12.31	22.95
8	54.57	4.90	43.24	7.65	35.68	11.02	26.23	19.58	20.56	30.60
10	81.23	6.12	64.39	9.56	53.17	13.77	39.14	24.48	30.72	38.25
13	130.17	7.96	103.26	12.43	85.32	17.90	62.90	31.82	49.44	49.73
16	189.60	9.79	150.49	15.30	124.41	22.03	91.82	39.17	72.26	61.20
20	284.81	12.24	226.19	19.13	187.10	27.54	138.24	48.96	108.92	76.50
25	429.08	15.30	340.93	23.91	282.17	34.43	208.71	61.20	164.64	95.63
30	600.90	18.36	477.65	28.69	395.48	41.31	292.79	73.44	231.16	114.75
35	799.95	21.42	636.09	33.47	526.84	48.20	390.32	85.68	308.39	133.88
40	1026.	24.48	816.02	38.26	676.08	55.08	501.16	97.92	396.21	153.00
45	1279.	27.54	1017.	43.04	843.03	61.97	625.23	110.16	494.54	172.13
50	1558.	30.60	1240.	47.82	1028.	68.85	762.44	122.40	603.35	191.25
55	1863.	33.66	1483.	52.60	1230.	75.74	912.71	134.64	722.58	210.38
60	2195.	36.72	1748.	57.38	1449.	82.62	1076.	146.88	852.11	229.50
65	2553.	39.78	2033.	62.17	1686.	89.51	1252.	159.12	992.01	248.63

Digitized by Google

LOSS OF HEAD DUE TO FRICTION OF WATER FLOWING IN PIPES 100 FEET IN LENGTH HAVING VERY SMOOTH INTERIOR SIDES.

(*Weston's Formula.*)

Velocity in Feet per Second.	Internal Diameter of Pipe in Inches.									
	1½		2		2½		3		3½	
	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.	Loss of Head due to Friction.	Gallons per Minute.
1	0.46	5.51	0.32	9.79	0.24	15.30	0.18	22.03	0.14	30.00
2	1.47	11.02	1.04	19.58	0.78	30.60	0.60	44.06	0.48	60.00
3	2.96	16.52	2.10	29.38	1.58	45.90	1.24	66.10	0.99	90.00
4	4.89	22.03	3.48	39.17	2.64	61.20	2.07	88.13	1.67	120.00
6	10.03	33.05	7.18	58.75	5.47	91.80	4.33	132.19	3.51	180.00
8	16.78	44.06	12.06	78.34	9.23	122.40	7.34	176.26	5.99	240.00
10	25.11	55.08	18.09	97.92	13.89	153.00	11.08	220.32	9.08	300.00
13	40.47	71.60	29.26	127.30	22.54	198.90	18.05	286.42	14.85	390.00
16	59.22	88.13	42.92	156.67	33.14	244.80	26.63	352.51	21.97	480.00
20	89.38	110.16	64.95	195.84	50.29	306.00	40.52	440.64	33.54	600.00
25	135.25	137.70	98.53	244.80	76.49	382.50	61.80	550.80	51.30	750.00
30	190.08	165.24	138.73	293.76	107.92	459.00	87.38	660.96	72.71	900.00
35	253.77	192.78	185.50	342.72	144.54	535.50	117.23	771.12	97.72	1050.00
40	326.24	220.32	238.78	391.68	186.31	612.00	151.32	881.28	126.33	1200.00
45	407.44	247.86	298.54	440.64	235.20	688.50	189.64	991.44	158.53	1350.00
50	497.30	275.40	364.73	489.60	285.19	765.00	232.15	1101.60	194.28	1500.00
55	596.79	302.94	437.33	538.56	342.26	841.50	278.87	1211.76	233.60	1650.00
60	702.87	330.48	516.31	587.52	404.38	918.00	329.76	1321.92	276.46	1800.00
65	818.51	358.02	601.66	636.48	471.55	994.50	384.82	1432.08	322.86	1950.00

DISCUSSION.

RUDOLPH HERING, M. Am. Soc. C. E.—The present paper, like the former ones of Mr. Weston, contains matter of considerable value and interest and bears testimony to the painstaking and the accuracy in details with which its author has approached the problem. The completeness with which most of the known experiments on the flow of water in pipes have been given and arranged is, I think, not exceeded in any other single publication. The new formula which he has developed therefrom for the flow in very smooth pipes of ¼ to 3½ inches in diameter is, as he clearly demonstrates, practically correct, and is possibly the best one that has up to the present time been suggested within the limits and under the qualifications given.

My object in sending this discussion is not to question the accuracy of this formula, but to express my doubts as to the propriety of advocating one having what I think is a faulty construction. Had the author

Digitized by Google

confined himself to recommending the table on page 55, which gives the values of the co-efficient of friction within the indicated limits of the diameter of pipe and of the velocity in the same, or had he recommended a diagram exhibiting these values, I should have said that I believe the table or diagram gives the most reliable published average results for the purpose set forth in its heading. To suggest a new formula, however, particularly with so limited a range of applicability as in this case, is, I think, somewhat questionable wisdom. It is a case similar to that mentioned by Mr. F. P. Stearns, M. Am. Soc. C. E., in the discussion of Yonge's formula (see *Transactions*, Am. Soc. C. E., Volume XIV, page 15), which if applied not far outside of a given range may lead far from the truth. Could we always append to the formula the limits of its range as readily as we can indicate them in the integral calculus, this objection to the formula would not hold good. But as a rule this is not done, and we find one book after another, or one engineer after another, quoting formulas, but stripped of some of their essential qualifications, and then using them for what they were never intended. By naturally fixing the limitations in the form of tables or diagrams no one could unsuspectingly drop into an error.

Further, I am not able to agree with Mr. Weston on the fundamental construction of his formula. He has followed the precepts of Darcy in recommending a separate and distinct mathematical expression for different degrees of roughness of the interior surface, whereas we observe as much and even more continuity in its variation, as it changes from the smoothness of glass to the roughness of old cast-iron pipe, than we observe in the diameter of marketable pipes, and therefore might, with almost equal propriety, deduce a separate formula for each diameter. In fact, we see that often a barely appreciable difference in the character of the interior surface has a greater effect upon the co-efficient than a measurable difference of diameter.

It therefore seems to me essential to the proper construction of any formula used for the flow of water in pipes, that the degree of roughness should be embodied in it as a variable and determinable quantity just as much as the diameter, the length of pipe, or its hydraulic gradient. The introduction of such a variable, as is well known, was first attempted by Messrs. Ganguillet and Kutter. While I am convinced that their own formula can be improved both in construction and in the numerical values of its constants, yet it seems indisputable that the suggestion of a variable co-efficient of roughness is a step in advance which I think should no longer be ignored. Such a co-efficient gives the engineer an opportunity to exercise practical judgment in the selection of a definite value for it in accordance with the nature of the perimeter, which he is incapable of doing intelligently with a formula in which the co-efficient represents the combined effects of a number of causes, and conceals the separate effect of each one of them.

Digitized by Google

Nor do I think it logical, by following Weisbach's lead, to base the variation of the only co-efficient upon the velocity rather than upon the gradient or slope which produces it. There are good reasons for not doing this in large rivers where the slope is often inconsistent with the velocity, or cannot be determined with equal accuracy. But for pipe systems, where the hydraulic gradient can so readily be measured, I think formulas ought to have their co-efficient based upon the slope, irrespective of the other inconvenience in Weisbach's method of being first obliged to approximate the desired velocity in order to find the proper coefficient.

That Mr. Weston could not obtain a satisfactory formula for old cast-iron pipes is due to the fact that virtually he was trying to find for them a fixed degree of roughness. Had he introduced a coefficient permitting the exercise of some judgment regarding the relative degree of this roughness in a particular pipe, he would, I think, have found a formula in which he could have had more confidence.

EDMUND B. WESTON, M. Am. Soc. C. E.—I regret that I have not been able to construct a formula for the flow of water in pipes having very smooth interior sides, similar to lead and brass pipes, which I can recommend for larger diameters than 3.5 inches; but unfortunately I had not sufficient experimental data to do this satisfactorily. I therefore restricted myself to the experimental data that I had at hand, and constructed a formula which agrees remarkably well with experimental results, as can be seen by examining the diagrams from No. 7 to No. 14. I was well aware that there would be a possibility of the formula being applied to pipes having larger diameters than 3.5 inches, consequently I was very particular in my paper to emphasize, in addition to the limit of diameters, the nature of the pipe for which the formula was intended. And as the pipes whose interior sides are very smooth, which have diameters larger than 3.5 inches, are not used at the present time to convey water, unless possibly in very exceptional cases, it seems to me that with reasonable precautions there should not be any danger of the formula being applied beyond its limit. In fact there are few formulas but what are in reality more or less limited in their range, if the experiments from which they were constructed are criterions; even Ganguillet and Kutter state that their formula must not be expected to apply to cases beyond the range of the data from which it was derived.

I recognize how valuable a reliable general formula would be which could be applied to all kinds of pipes, and which would contain a co-efficient permitting the exercise of some judgment regarding the relative degree of roughness in a particular pipe; but had I been disposed to have attempted the construction of such a formula, it would not have been possible for me to have done so from the experimental data that I had accumulated, as a preliminary investigation that I

Digitized by Google

made relative to a formula of this kind developed results from these data which were entirely too conflicting.

Mr. Hering's opinion that formulas ought to have their co-efficients based upon the hydraulic gradient rather than the velocity, does not entirely coincide with my views in regard to formulas for the flow of water in pipes under pressure, although I admit that such formulas would be very convenient at times, as in the majority of cases in water-pipe calculations the result sought after is the loss of head due to friction, and not the velocity, as is generally the case in calculations relating to rivers and open channels.

In regard to constructing a perfect and reliable general formula that could be applied to old cast-iron pipes, if such a thing were possible, especially if the age of the pipes was the basis upon which judgment was to be formed as to the condition of their interior surfaces, it seems to me that in addition to a co-efficient of roughness, etc., etc., it would be quite important to embody in the formula a co-efficient dependent upon the chemical constituents of the water that was flowing in the pipes, or that was to flow in the same, as it is well known that some waters cause cast-iron pipes to corrode much faster than other waters. For instance, it can be seen in Table No. 1 that co-efficients of friction (ζ) worked out from experiments that were made in Philadelphia with cast-iron pipes respectively seven and eleven years old, are about twice as large as co-efficients of friction (ζ) derived from experiments that were made in England with cast-iron pipes respectively seven and thirteen years old.

CHAS. E. EMERY, M. Am. Soc. C. E.—I had occasion to closely examine the general formulas on the subject of the flow of fluids in developing an expression for the flow of steam. The formula selected was based on that of Weisbach, and the simple form applicable for water was found to answer every purpose for small losses of pressure in transmission, although a differential formula was developed to cover all conditions. In a cursory examination of the various papers that have been read on the subject of the flow of water, it has occurred to me that the consideration of one important source of information has been omitted. Experiments have been made with great care on the skin resistance of vessels moving through the water. Froude published, in the Transactions of the British Association for the Advancement of Science, some ten years ago, a series of curves showing the resistance of a large model hull at different velocities and for different lengths of surface operated upon. He found that the unit resistance decreased rapidly as the length was increased, but became nearly constant after the length exceeded 100 feet. Necessarily the frictional resistance, caused by the interior surface of pipes, corresponds to the skin resistance of a vessel in the water, and the results from the two methods of experiment can

Digitized by Google

be compared with advantage. The Froude results show that experiments with short pipes should show greater resistance.

I fully agree with Mr. Hering that all the results of experiments should be shown by one general formula. This can only be accomplished by introducing in such formula a suitable number of variables, usually called constants, which can be modified to suit the particular conditions and thereby cause the formula to produce substantially the experimental results. It has been stated again and again that the character of the interior surface should be considered in the formula, but it does not as yet appear to have been done satisfactorily. I have heard the proposed constant termed the co-efficient of rugosity, although the term does not appear in either of the papers of the Society. The skin resistance has been pretty well shown to be due to eddies produced by the roughness of the surface. Old navigators can tell pretty well the speed of a vessel by looking at the distance the foam produced by the eddies extends from the side. In a running stream the water striking large stones below is thrown upward to the surface, as shown clearly by Mr. Francis' experiments with white-wash in one of the races at Lowell. In a discussion of that paper I called attention to how well these experiments compared with the accepted theories of eddy resistance. The rougher the surface the more the particles are deflected from a direct course and the farther the eddy resistance penetrates the central stream. It would seem that sufficient experiments are now available with surfaces of different characters to enable a co-efficient to be introduced into the ordinary water formula which would allow accurately for all customary variations. It might be well for those interested in the subject to examine some discussions by Rankin and others on the laws of eddy resistance, so that the co-efficient of rugosity would be introduced at the proper power to produce variations corresponding with those shown by experiment.

Mr. WESTON.—D'Aubuisson in his "Traité D'Hydraulique," published, I think, about 1850, in an article upon the action of water by its resistance, recognizes that the increase in the length of a body moving through the water very perceivably diminishes its unit of resistance.

A few preliminary experiments that I made in 1876, relative to the flow of water in pipes, owing to the peculiarity of the results obtained, led me to make an investigation with the experimental data of quite a number of authorities in order to ascertain if the co-efficient of friction (ζ) was in any way dependent upon the length of the pipes. This investigation was rather an unsatisfactory one in some respects, as in nearly the whole of the experimental data of the very short pipes that were used the co-efficient of influx (ε) entered into the problem, and in some instances it had to be assumed. The results, however, showed

Digitized by Google

decidedly that the value of the co-efficient of friction (ζ) was not influenced by the length of the pipes.

I have already expressed my views in regard to a general formula in my reply to Mr. Hering's discussion.

E. SHERMAN GOULD, M. Am. Soc. C. E.—This paper contains the results of some interesting experiments upon the flow of water in pipes, as well as a very comprehensive and valuable review of the labors of other investigators in the same line.

The great number of these experiments which have been made, and the somewhat discordant formulas deduced from them, naturally lead us to inquire into the cause of such disagreement.

This much must be said of all formulas founded upon experiment: granting the accuracy of the measurements and records, the formulas must be true for the cases from which they were derived, and any others which should give different results would, for these particular cases, be incorrect, for the results were known in advance, and the formulas made to fit them.

The value of such formulas depends wholly upon the extent, perfection and trustworthiness of their experimental data. In these respects the magnificent researches of Darcy seem to command the greatest confidence. No other experiments, unless conducted upon a still more perfect scale and guided by a surer and more discriminating judgment than his, should be considered as invalidating the results left us by this illustrious experimentalist. So far from being invalidated, it would appear that his formulas have been generally corroborated by subsequent investigations. Mr. Charles B. Brush's observations on a 20-inch main, which have been already laid before the Society, and quoted in this paper, agreed very closely with those obtained by using Darcy's formulas for the same data. Darcy's experiments do not include the larger diameters of pipe, but I understand that the actual volumes discharged by the 48-inch main of the Bronx River Water Supply agree within three per cent. of those given by his formula. I am glad to perceive, by the paper now under discussion, that Mr. Weston also finds his results to agree very well with those of Darcy.

From a practical point of view, the state of the question seems to be this: Given a certain number of trustworthy experiments it is easy to formulate an equation which shall return the observed results, and this equation will always be true for identical conditions of pipe. But any change in the conditions will vitiate the formula to a greater or less extent, and render a new one necessary. In order to apply the formula, we must know exactly the conditions upon which it was framed. These can only be described by saying that the interior surface of the pipe was "smooth," or "rough," or "slightly tuberculated," etc. All this is very vague, and yet the state of the inside surface is a controlling factor in the problem. Moreover, a system of pipes when first laid is, or

Digitized by Google

should be, in a condition of smoothness. Later, it will become more or less rough or even tuberculated, which will not only change the character of the inside surface, but also reduce the diameter. It would seem idle, therefore, to go into a great refinement in the matter of a formula, particularly if the refinement leads to a shaving down of the diameter.

As regards the effect of the velocity itself as affecting the formula, although there seems to be no doubt that the co-efficient for the same pipe varies somewhat with the velocity—that is, with the sine of the angle of inclination—yet, it must be remembered that in practice no very great extremes of velocity, either way, are admissible, so that this factor is kept within comparatively narrow limits of variation, admitting of a mean value being assumed, for most cases.

I would not be understood as depreciating the value of such researches as those embodied in Mr. Weston's able and instructive paper. They lie at the very foundation of all sound science. I would wish simply to direct attention to the immediate bearing of the formulated results upon the actual practice of designing a line of water supply pipes, and to indicate what we can and what we cannot expect from a formula in this connection.

Mr. Weston.—That Mr. Gould has such a high opinion of the value of the investigation and formulas of the late Henry Darcy, I am very glad to learn, not only on account of my having recommended his formulas for new cast-iron pipes, but for the reason that a number of writers upon hydraulic subjects have seemingly taken great pains not only to depreciate the formulas of this eminent engineer, but also to question the value of his experimental results. I regret, however, that Mr. Darcy did not deduce a formula for pipes having very smooth interior sides, although his experiments with pipes of this class were somewhat limited.

Mansfield Merriman, M. Am. Soc. C. E.—The formula deduced by Mr. Weston for loss of head in friction has the disadvantage, in common with those presented by other authors, of being empirical and not rational. It is not easy to see, however, how a theoretical expression for this lost head can be established, and accordingly experiment must be the only guide for a long time to come. Mr. Weston's formula can be written thus:

$$h_f = 0.0126 \frac{l}{d} \cdot \frac{v^2}{2g} + 0.0315 \frac{l}{d} \cdot \frac{v^{\frac{3}{2}}}{2g} - 0.06\ l\ \frac{v^{\frac{3}{2}}}{2g}$$

in which only the first term of the second member is homogeneous with h_f, indicating that the numbers 0.0315 and 0.06 are functions of v or d, whose relations are not yet known. I do not like the presence of the minus sign before the last term of this expression, for if the friction head be due to several causes it is not easy to see why any cause would produce a negative effect.

Digitized by Google

The diagrams show that the formula agrees very well with the experiments, and hence the labors of Mr. Weston will be of value to all who wish to make computations on the discharge through pipes. It is my impression that other formulas might be devised which would exhibit an accordance nearly or quite as good, but it does not seem very important that further investigations should be undertaken until something is known regarding the theoretical laws which control the friction of flowing water. With such a complex empirical expression as that of Kutter's formula I have no sympathy whatever. The laws of nature are, of course, not simple, but it cannot be at all supposed that the true law of loss of energy in friction is represented by such algebra as Kutter's formula contains.

The loss of head due to sudden contraction of the diameter of a pipe is, I think, always less than that given by Mr. Weston. He states for all cases that the co-efficient ϕ is 0.316, and that this is increased by resistance and friction to 0.505. Now the number 0.505 applies to the case where the diameter d (Fig. 4),* is very large compared with d_1, as for instance, when water enters a pipe from a reservoir. It is clear that when $d = d_1$, the value of ϕ should be zero, for in this event no loss of head due to contraction occurs. A proper expression for the value of ϕ should hence depend upon the ratio of d to d_1, As a reasonable approximation I suggest the formula:

$$\phi = \tfrac{1}{3}\left(\frac{1}{c^1} - 1\right)^2$$

in which c^1 is the co-efficient of contraction of the stream in passing through the orifice whose diameter is d_1. Let r denote the ratio of d to d_1, then the value of c^1 in terms of r may be stated by the following expression, proposed in the second edition of my Treatise on Hydraulics:

$$c^1 = 0.582 + \frac{0.0418}{1.1 - r}$$

which is based upon the mean value $c^1 = 0.62$ when $r = 0$, and the known result $c^1 = 1.0$ when $r = 1$. The numerical values of ϕ for different values of r can now be ascertained, c^1 being first computed, and thus are found,

For $r =$	0.0,	0.2,	0.4,	0.6,	0.7,	0.8,	0.9,	1.0.
$\phi =$	0.50,	0.47,	0.42,	0.34,	0.28,	0.20,	0.09,	0.00.

These values of ϕ are given with only two decimals, for I regard it as useless to write the third decimal in cases where even the second is uncertain.

Mr. WESTON.—I consider the ideas of Professor Merriman relative to the co-efficient of resistance (ϕ) of water passing from a large to a small pipe very reasonable. I gave in my paper, as I so stated, the co-efficient

* d and d_1 being diameters, and F and F_1 areas, as indicated on Fig. 4, page 62.

Digitized by Google

ϕ of Weisbach, as I had not made any experimental investigation based upon the idea of determining a co-efficient of this kind, considering that his co-efficient was a safe and simple one to use. The difference, however, in the loss of head by the formula $h_f = \phi \frac{v^2}{2g}$, would be only 0.11 feet for the greatest velocity that water should be allowed to flow through distribution pipes, by using either the smallest co-efficient 0.09 given by Professor Merriman, or the co-efficient 0.505 of Weisbach.

I have found recently in the second volume of Weisbach's work that a formula corresponding to $\phi = \left(1 - \frac{F_1}{F}\right)^2$ is mentioned (the co-efficient ϕ given in my paper being taken from the first volume).

JOHN R. FREEMAN, M. Am. Soc. C. E.—The present state of the art of computing the loss of head in an ordinary given pipe when delivering a given quantity, is about like figuring out a formula to four places of decimals and getting a result that may be relied upon as correct within 25 per cent. We are all interested in efforts to reduce this margin of uncertainty.

Occasional reports of Mr. Weston's work in the field of water supply have given me full confidence in the earnestness with which he seeks the truth and have led me to respect the generous diligence with which he gives the results of his labors to the craft.

I feel more free to discuss the matter at this time from the fact that the author has full opportunity to reply to all comments and criticisms in closing the discussion, and believe it for the best interests of both the Society and the authors that all papers like this should be subject to a searching examination from the different points of view of different minds while the whole matter is fresh and the author has opportunity to answer and explain.

It seems to me that Mr. Weston is off the true road towards a thoroughly satisfactory formula for the flow of water in pipes.

I would maintain as a foundation principle that a formula representing the law of flow in pipes, to be satisfactory, must contain as a very important and controlling factor, a term dependent upon the roughness of the interior surface of the pipe.

Mr. Weston's discussion of this question does this only in a crude and indirect manner, by adapting special formulas, each of narrow application, to some of the different classes of pipes. In one case his variable co-efficient is a function of only the diameter as a variable, in another of only the velocity, and in another of both the diameter and the velocity.

There is no reason to suppose that the actual law thus varies.

His investigation brings but little new data to light, and thus is not an experimental but an algebraic research, seeking the equation to a curve, which shall be as nearly as possible the mean of the plotted data

Digitized by Google

for each of the several classes into which he divides certain data already well known and discussed.

Mr. Weston's form of investigation does not seek an expression for a law that is broad and general and covers the flow in all pipes, and in fairness to him we must, I think, admit that there is not on record the data with which he could have conveniently sought one.

I believe there is such a law, and that when we have more experiments it will be found. The missing link now needed to tie experiments on different pipes together and bring them exactly into one line, or explain the discrepancies in our data such as are so well shown in the convenient diagrams appended to Mr. Weston's paper, is an accurate and strictly definite description of the character of the surface.

It is surprising how this part of the description has been neglected in nearly all recorded experiments relating to the friction loss in iron pipes. Take even Darcy's experiments, in which he demonstrated what Hamilton Smith, Jr. calls the most brilliant hydraulic discovery of the century—the powerful influence of character of surface on the flow. The part of this research relating to effect of rough deposits was a qualitative analysis and not a quantitative analysis, for where, in his memoir, can you find it stated whether the tubercles or deposits which he found to double the friction loss were on an average as large as grains of wheat, or as large as cherries? This is important information to have, since it might correspond to 50 per cent. or more difference in the loss of head.

While discussing the desirability of including a special co-efficient of roughness in our future formula, we may refer to the fact that that equation of wonderful range, the "Kutter formula," is applicable to express the law of flow in pipes, and that it contains a special co-efficient of roughness, and that in the very complete and convenient table of hydraulic data, given by Messrs. Hering and Trautwine in their appendix to Ganguillet and Kutter's treatise,* published about a year ago, nearly all of the five hundred experiments tabulated and described by Mr. Weston are there presented with the Kutter co-efficient of roughness computed for each.

This effort to describe the law of flow in all pipes by a single equation is highly commendable from a broad, general point of view, but it is not, however, altogether satisfactory.

I am inclined to think that some far simpler equation than that of the Kutter co-efficient may be found, which will express the law of flow in pipes so accurately that the error coming from approximations involved in its algebraic form will be le s than the actual variations in flow due to differences of roughness so small as to be altogether inappreciable in any ordinary description in character of surface.

---

* Flow of Water in Rivers and other Channels. Wiley & Sons. New York, 1889.

Digitized by Google

Turning, now, to the subject of loss of head in clean and new cast-iron pipes: These by no means all have the same character, and it is useless to expect one formula with one constant to apply to all. Thus, for instance, we, perhaps, explain in part the difference of 25 per cent. in co-efficient ζ found between Darcy's results and the two newly reported experiments of Mr. Weston on a 6-inch pipe. I have seen different lots of new 6-inch cast-iron pipe, which differed so much in character of surface that I have little doubt but the loss of head would be considerably more than 25 per cent. greater in one than in another. This difference in surface came from the degree of skill with which the tar coating was applied even more than from the difference in the casting as it came from the foundry sand. Thus some tar-coated cast-iron pipe is very smooth on the inside, while other pipe has "ribs" of tar projecting about $\frac{2}{16}$-inch, or even $\frac{1}{4}$-inch, running part way around the circumference where the tar has run while soft. On studying the collection of experiments on new cast-iron pipe now available as data and transcribed from one book to another, and given with fullness in Iben's, Smith's, Hering and Trautwine's and Weston's discussions, it will be seen that causes like this can easily obscure the true law or the refinements of a complicated formula.

Until we have new experiments on pipes in which great care is taken to accurately and definitely describe the exact degree of smoothness of the interior surface, it is almost hopeless to try and derive an accurate general formula. Meanwhile, had we not best try earnestly to avoid complicated algebraic expressions, and keep our formulas in the simplest possible form.

In other words, is not that simple formula of the last century, the Chezy formula, so called, when accompanied by a short and simple table or diagram of values for its co-efficient, better for practical use than any of the more complicated formulas? Until we get that extended series of quantitative experiments on effect of roughness is not this simple Chezy formula: $V = C\sqrt{RS}$.

In which V = velocity in feet per second.
C = co-efficient of flow dependent on diameter, velocity and roughness (Smith represents this by N).
R = hydraulic radius = $\frac{1}{4}$ diameter of pipes of circular section.
S = slope = ratio of loss of head to length of pipe,

the best framework on which to display for comparison such new bits of experimental data as we may from time to time secure. Hamilton Smith, Jr., so used it in his most excellent treatise * (which is, perhaps, the most thorough of all in the care taken to exclude questionable data); and so did Hering and Trautwine in the appendix to their excellent

* Hydraulics, Hamilton Smith, Jr. Wiley & Sons. New York, 1886.

Digitized by Google

translation of the Ganguillet and Kutter work, given to the profession a year ago.

The valuable tables of data in the two recent books just mentioned give nearly all the experiments on flow in pipes recorded for a hundred years back, which have value as data and include nearly all those presented by Mr. Weston. When presenting new data by reducing it to this form, it is much more convenient for reference and comparison.

For the present, is it not better to observe experiment and secure new co-efficients for the old formula rather than re-thrash old, and in many cases blighted straw, to devise a new formula?

With regard to very rough or corroded pipes, Mr. Weston, like nearly every author who attempts to devise special formulas for the experiments now on record, or attempts to make experiments on one pipe fall in line with those on another, finally gives it up as a bad job.

The trouble in reconciling the experiments is, I think, in general, not that the loss of head has been inaccurately measured, not that the delivery was improperly determined, nor is it that the net diameter is not precisely known.

The uncertainty comes from vagueness in describing the character of the surface. The exact character of the surface is not easy to determine. In many cases there is no opportunity to inspect it, and even if opened to inspection a corroded surface is difficult to describe with precision.

In the present state of our experimental knowledge is it not almost useless to attempt a special form of equation for these very rough pipes? Is it not better to arrange the results of such experiments in the form of a comparison of the loss of head found in the rough pipe with that deduced from our simple formula for an average ordinarily smooth pipe, and thus, for instance, say that in the case of a certain pipe corroded as described, it was found that the loss of pressure was 2.4 times as great as the formula shows for a smooth pipe of same size and with same delivery? With each experiment expressed in this way, but condensed into tabular form along with the various hydraulic elements of the case, the present data is in most convenient shape for practical application, and is of very great value in conspicuously calling attention to the danger when designing works of computing the friction loss as though the pipes were always to remain new and clean.

With regard to the particular style for the fundamental formulas, I notice that Mr. Weston, on page 6 of his paper, presents the formulas which his own experience has shown most convenient. These are those sometimes known as Professor Weisbach's.* Is not the convenience of this type, as compared with the modified Chezy form as used by Smith, Hering and Trautwine and others, a question of personal habit more

* Cox's Translation Weisbach's Mechanics, page 866 and page 870. Van Nostrand, 1872.

Digitized by Google

than anything else? The latter formulas include the value of $\sqrt{2g}$ in its constant, and thus gain slightly in simplicity. I was interested a while ago to look up this matter and see how much accuracy it was possible to lose in so doing.

If for the value of $\sqrt{2g}$ we always use 8.02, the greatest possible error that it will involve for any place in latitudes anywhere between the southern point of Greenland and Key West, and anywhere from sea level to 5 000 feet elevation will not be more than one-tenth of 1 per cent.

Thus, proper as it may seem to keep this value in sight in a general formula for text-book use, it seems needless to load a practical formula down with it, and we are justified in incorporating merely its average value into the constant of our formula for ready practical use where differences of surface so small they can hardly be noticed may cause 5 per cent. difference in the friction loss.*

On pages 6 and 7 of Mr. Weston's paper certain special formulas are presented for compound pipes and branching pipes. These are presented for practical use, and Mr. Weston uses their derivation as an argument for preferring the Weisbach arrangement of formula. On looking into these with a little care it appears that the formulas are less simple in their application than would appear from the first glance, since when seeking v for a given h in problem No. 5, for instance, the several values of ζ all depend on v, and the problem can therefore be solved only by successive approximations. Second, it seems to me that problem No. 5 especially presents a symmetry in the equal size and length of the pipes in the successive subdivisions, which would almost never occur in ordinary practice, and therefore I do not think the facility in deriving these formulas should be allowed to weigh too heavily in favor of the Weisbach form.

On pages 62 and 63 Mr. Weston copies from Weisbach without change or comment some formulas for effect of curves, which, though sad to tell, are the best yet on record, were derived from very small pipes, and may with good reason be distrusted, and ought not to be quoted in this manner without some limit or qualifications.

It may be noted that at the middle of page 62 the formula for effect of contraction as it stands is without meaning. A proper form is $\phi = \left(\frac{1}{c} - 1\right)^2$ in which c is co-efficient of contraction which depends on the relative area of F_1 and F, and thus needs a table of values to accompany the formula.

In other words, the formula as printed implies that ϕ is a constant whatever the amount of reduction in diameter; whereas, from the nature of the case, it must be a variable of considerable range.

* We might, indeed, in the effort to get things into the most convenient shape for practical use waive even a little more of the theoretical shape and write $Q = c' \sqrt{d^5 \frac{h}{l}}$.

Digitized by Google

With regard to the table of data and descriptions, occupying from the 8th to the 48th page, it appears that the experiments on two of the pipes there mentioned are original and published for the first time. Nearly all the others, which are reliable enough to found a formula upon, have been discussed in what seems to me a more thorough and searching manner by a distinguished member of our Society, Mr. Hamilton Smith, Jr., in his book published three years ago, and, as already intimated, all have their hydraulic elements and co-efficients adapted for both the Kutter and the Chezy formulas presented in the appendix to Hering and Trautwine's translation of Kutter. It is a matter to be regretted that the loss of flow deduced by Hamilton Smith, Jr., from substantially the same experiments discussed by Weston, was not represented along with the other curves upon the very excellent and convenient diagrams of Mr. Weston. This, prior to Mr. Weston, was the most recent study of the subject, and although Mr. Smith leaves the subject in rather an abrupt and incomplete shape, in that his values for $\triangle$ are not clearly defined, and that the gaps between his curves of values for practical use are rather wide, especially for pipes under 12 inches in diameter—(this was influenced, no doubt, by a lack of data, which still exists)—yet, his conclusions are of great value.

Mr. Smith used substantially the same data as that from which Mr. Weston satisfied himself that the Darcy formulas were satisfactory, and came to a different conclusion and presented a law of his own derivation, and one which, like Mr. Weston's new formula for smooth pipes, made the value of the co-efficient depend on both the diameter and the velocity. It is true Mr. Smith presented his law expressing the values of the co-efficient in graphical form rather than in an algebraic one, with perhaps the same view as expressed by a previous speaker, that a curve or a table is less liable to be used beyond its proper limits than a formula.

We may also refer to the very convenient diagram given in Mr. J. T. Fanning's comments on Mr. Brush's very interesting paper on Waste and Friction Loss in Water Mains,* as another illustration of the use of a diagram for expressing the various values for the co-efficient of flow in terms of both the diameter and velocity.

Finally, I may venture the comment that so far as I yet see Mr. Weston has left the main question just where he found it; that is the question concerning the flow in ordinary cast-iron pipes, which are those that the hydraulic engineer has to deal with in nineteen cases out of twenty. As I read the paper through, it seemed that the main new feature of value presented is that of the formula for very smooth pipes, from $\frac{1}{4}$ to $3\frac{1}{4}$ inches in diameter.

The presentation of the experiments by Mr. Weston upon the 1-inch

* *Transactions*, Vol. XIX, No. 395, p. 112, September, 1888.

Digitized by Google

tin-lined iron pipe and the 6-inch cast-iron pipe are to be welcomed; but it is to be regretted that the possible limits of error are not investigated and stated, and the experiments described with the fullness so necessary to give high scientific value—such, for instance, as that with which Hamilton Smith, Jr., describes most of the new data on pipes which he presented to the Society in 1883, and described in his Hydraulics, page 290 *et seq.* As the record now stands we must regard these new experiments merely as approximations.

In the final revision of his paper, I hope Mr. Weston may explain a little more fully the magnitude of the obstruction formed by the sheet tin bushings at each joint, and will give more fully the method employed for determining the loss in the ¼ tap and in the main from the reservoir.

As to the 6-inch pipe it would add to the value of the experiments if Mr. Weston would kindly state the possible limits of error in the following directions:

(*a.*) In general even the best Bourdon gauges are not instruments of precision, and scales are liable to unaccountable derangements. Were these gauges tested by a mercury column immediately before and after the experiments, or was their position reversed? An error of ¼ pound to each gauge alone might make experiment 325 11 per cent. in error.

(*b.*) Is it certain piezometer tubes were free of air?

(*c.*) Were piezometric orifices exactly flush with inner surface and normal to axis of pipe?

(*d.*) Was diameter of this particular pipe accurately measured, or is the 6 inches the "foundry size?"

(*e.*) This pipe apparently was not laid with a view to the experiments or inspected after them. Is it absolutely certain it contained no obstruction like a bunch of lead from a bad joint, or that in the four years' use no tuberculation whatever had taken place?

(*f.*) Were the lengths of hose the same ones that had been gauged, or were they other pieces subject to the ordinary commercial variations in diameter and character of surface of different lots?

Eternal vigilance and everlasting patience are the price of scientific accuracy in this class of work.

These questions are of interest since Mr. Weston finds a friction loss about 20 or 25 per cent. greater than that found by Darcy for the cast-iron pipe nearest to this in diameter. Nevertheless, even if these questions cannot be answered, and if the experiments are less precise than Mr. Weston would have gladly made had circumstances favored, they are of value and interest as data, and all such are to be welcomed as a practical guide until the longed-for elaborate series of experiments are made by a second Darcy.

The experiments selected by Mr. Weston in developing his formula

Digitized by Google

for smooth pipes are, in some respects, questionable, though undoubtedly they were the best data to be found, and it is no fault of his that better data were not in existence.

Thus: *First.*—Is it not a little doubtful to use experiments 1 to 14 as a basis for a formula, without some statement as to length of pipe and conditions under which experiments were made? Mr. Smith and Messrs. Hering and Trautwine exclude these from their table of data.

Second.—In making use of experiments 28 and 29 by Rennie, upon a pipe only 15 feet long, and experiments 59 and 60 by Neville, there is liability to error from the fact that in such very short pipes the distribution of velocity and regimen of flow is not fully established at upper portion of pipe. This may be illustrated by reference to Series III and N, page 239, Smith's Hydraulics. Moreover, in each of these cases of Rennie's and Neville's experiments, the co-efficient of influx which has to be guessed at exercises a proportionally larger influence on the total loss of head than for longer pipes.

Third.—For exact scientific purposes it seems hardly fair to class on the ground of smoothness experiments on new wrought-iron lap-welded gas pipe, ¾-inch and 1-inch diameter, along with the pipes of glass and lead. Is it not uncommon for iron pipe to come from the welding furnace with a skin as smooth as that on a lead pipe as it comes from the die? Moreover, why from description alone is it proper to make use of Smith's experiments on new uncoated 1-inch gas pipe and reject Darcy's experiments on new 1-inch gas pipe, even though results do not agree?

Passing to the pipes of larger diameters, quoted by Mr. Weston, if the 3¼-sheet-iron pipe and also the 7.40-inch and the 11.69-inch of Darcy had riveted seams, as stated by Hering and Trautwine, and as seems natural, it certainly would give a much greater friction loss than a smooth lead pipe, and cannot properly be included.

It is hardly following the true scientific method, but is arguing in a circle, to include experiment No. 58 by Hodson and No. 237 by Dr. Robinson, also Nos. 124 to 139 by Smeaton, as data for determining law of flow in very smooth pipes, merely because they happen to coincide approximately with the formula, while actually the smoothness of the pipe is not stated and the kind of pipe or manner of experiment is not known, and the diameter given by Mr. Weston to thousandths of an inch was apparently merely the nominal or commercial diameter.

Fourth.—For the tin pipe of experiments 146 to 160 we may ask, in order to gain an idea of the smoothness, just what kind of pipe was Bossut's "tin" pipe of one hundred years ago?

Fifth.—It may be of interest to note that the experiments of Leslie on 2½-inch lead pipe, No. 250 to 269, and of Provis, on 1¼-inch lead pipe (Nos. 175 to 211), though referred to by Mr. Smith, are by him excluded from his results as unreliable.

Sixth.—Excluding the questionable data, we see that the largest

Digitized by Google

smooth pipe left to furnish data for the new formula is the 2-inch glass pipe of Darcy.

Seventh.—To a person who is experienced in delicate and rigorously accurate hydraulic experimentation, the question will often occur, when trying to reconcile these early experiments, Did the experimenter clearly understand the disturbing influence of air bubbles, and were they driven out and excluded with positive certainty?

As we thus carefully examine the quality of the data we see how unsatisfactory much of it is as a foundation for a new formula for very smooth pipes, and how very few are the really reliable experiments.

If we wish accurate knowledge of the area of a sharply bounded field, is it not better to take a steel tape and a first-class modern transit and go out and survey it, making a clean, fresh, first-class job instead of hunting among the archives and averaging the more or less rough surveys of the past hundred years?

If, for the time being, we desire working knowledge more than ancient history, and need accurate and convenient knowledge of the laws of flow in very smooth pipes like drawn lead, then if time is money, it would cost very little more to have some good lead pipes made ¾ inch, ¾ inch, 1 inch, 1¼ inch, 2 inch and 3 inch, a hundred feet of each, handle it delicately, keep it straight and perfect as it comes from the die, experiment on it with velocities from ¼ foot to 25 feet per second, then try it bent around certain definite curves, let a plumber coil it up and straighten it out two or three times, crook it around half a dozen corners and then experiment on it again.

This is work which could, for instance, be very easily done in the new hydraulic laboratory now under construction for the Massachusetts Institute of Technology, and I sincerely hope it may be undertaken somewhere and the work done on a suitable scale and under conditions that may make the results unquestionable.

Then with the work once thoroughly well done, the results, with their own sharply defined limits, may stand unchallenged for a hundred years.

We all owe thanks to Mr. Weston for the diagrams appended to his paper, which show in an extremely convenient and interesting manner the way in which our available data agrees with various standard formulas.

To criticise the investigation or to detract from the just praise due Mr. Weston for his industry in presenting these matters is far from my purpose.

The main points I would make are:

First.—In securing the data now on record far too little attention has been paid by experimenters to ascertaining and describing the exact degree of smoothness of the wetted surface.

Second.—Investigators in attempting to derive formulas from the

Digitized by Google

data have given too little attention to incorporating in their formulas a factor dependent on this roughness.

Third.—The profession greatly needs a new and extended series of experiments before satisfactory general laws or formulas can be derived.

Fourth.—Until we get these it is better to make use of very simple and familiar formulas like that of Chezy, and devote our opportunities to extending, tabulating and classifying values of the co-efficient.

In closing I may say that my own experience goes to show that a table on the plan of the excellent little table published by George A. Ellis, C.E., about ten years ago, but constructed with smaller gaps between quantities and diameters, is more convenient for practical use than any formulas or the tables in any of the engineering pocket reference books.

The plan of this is as below:

Gallons discharged per minute.	4-Inch Pipe.		6-Inch Pipe.		Etc.
	Mean velocity. Feet per second.	Loss of head per 100 feet.	Mean velocity. Feet per second.	Loss of head per 100 feet.	

If appended to this we could have compiled a table of factors by which to multiply these losses of head to allow for different degrees of roughness, the whole would be of the utmost convenience, and I hope some one who is ready to take his pay in the gratitude of the profession will prepare such a table based on some standard investigation.

Mr. Weston.—I think it is fortunate in many respects that all of the members of the human race have not the same opinions, for if such was the case, there would be but little I fear to stimulate progress in the direction of making new discoveries and improvements. Therefore, in my reply to the somewhat lengthy discussion of Mr. Freeman, I shall endeavor to confine myself mainly to answering the direct questions that he has propounded, and to replying to a number of his adverse criticisms that I consider were based upon misapprehensions, which, if I have surmised correctly, may very probably have been owing to his not having had a sufficient length of time at his disposal to make a thorough examination of my paper.

Digitized by Google

In reply to the remarks of Mr. Freeman (page 74), which commence thus: "It seems to me that Mr. Weston is off the true road toward a thoroughly satisfactory formula," * * * * and end thus: * * * * "In one case his variable co-efficient is a function of only the diameter as a variable, in another of only the velocity, and in another of both the diameter and the velocity"—I will say: *First*—What I have already intimated in my reply to the discussion of Mr. Hering, viz.: that I was governed by the experimental data that I possessed, and that the first preliminary investigations that I made relative to constructing a formula were to see if one could not be formed that would be sufficiently broad and general in its application to cover all kinds of pipes, and which would contain a factor dependent upon the roughness of the interior surface of the pipe. I was soon convinced, however, that a reliable formula of this kind could not be constructed. *Second*—That it strikes me the application of the formulas that I have recommended are just the reverse of narrow, as one of the formulas can be applied to all pipes having very smooth interior sides, from ¼-inch to 3½ inches in diameter (pipes of this kind larger than 3½ inches being very rarely used for conveying water), and the other formula can be applied to all pipes having interior sides similar to new cast-iron pipes. *Third*—That a careful examination of my paper will show that I recommend the latter formula (Darcy's) on account of its conforming closely to experimental results, and for its simplicity and not for its particular form. *Fourth*—That the other formulas that I have constructed, that have been mentioned in my paper, were computed from a limited number of experimental results and were not recommended for general use.

In response to that portion of the discussion of Mr. Freeman (page 75), which commences thus: "While discussing the desirability of including a special co-efficient of roughness in our future formula," * * * * and ends thus: * * * * "Nearly all of the five hundred experiments tabulated and discussed by Mr. Weston are these tabulated with the Kutter co-efficient of roughness computed for each"—I will state that in my opinion there are reasons for thinking that it would be questionable to apply the "Kutter formula" to pipes in which water was flowing under pressure, as this formula was constructed from experiments that were made with water flowing in open channels. But assuming that in special cases "Kutter's formula" was adapted for the purpose, and that the tabulated experimental co-efficients of roughness mentioned by Mr. Freeman were available, the results that would be obtained by using these co-efficients in this complicated formula would not be more accurate, and the opportunities for exercising personal judgment as to the relative degree of roughness of the interior surface of the pipes more facilitated, than they would if Table No. 1 of my paper was referred to,

Digitized by Google

in which each class of experiment is specified, and an experimental co-efficient of friction (ζ) selected and used in the simple formula,

$$v = \sqrt{\frac{2gh}{\zeta \frac{s}{d}}}$$

In reply to Mr. Freeman's advocacy of the Chezy formula, I would call attention to pages 4, 5, 6 and 7 of my paper, where I discuss the form of formula the best adapted for the flow of water in pipes.

I will say in reply to the remarks of Mr. Freeman (page 78) relative to the forms of formulas that I have given on pages 6 and 7, that the values of the co-efficients of friction (ζ) that are included in these formulas do not necessarily depend upon the velocity (v), even in the first formula of each example which is arranged for computing the velocity for a given head, as co-efficients to friction (ζ) derived from "Darcy's formula," an expression for which I have given on pages 57 and 61, can be used and the velocity determined without approximation; also when high velocities enter into the problem, there will be but little need of approximating when using co-efficients of friction (ζ) dependent upon the velocities that have been determined from the majority of known formulas, as their values change very slowly after reaching a velocity of 5 feet per second. Then the second formula in each example is arranged for computing the loss of head due to friction (h_f), and can be directly solved; even if co-efficients of friction (ζ), dependent upon the velocity, are used as in these cases, the velocity in the outlet pipe or pipes is one of the known quantites, from which, in cases of branch or compound pipes, the velocities in the other pipes can be readily determined and co-efficients of friction (ζ) selected corresponding to them.

In respect to the co-efficients of resistance due to contraction (ϕ), mentioned by Mr. Freeman (page 78), my reply to Professor Merriman's discussion expresses my ideas upon the subject.

The reason why I did not plat upon the diagrams of my paper the results of the conclusions of Mr. Smith, given in his work referred to by Mr. Freeman (page 79), is, that my investigations were relative to formulas for the flow of water in pipes, and Mr. Smith only gives graphical results, and not formulas.

In reply to the statement of Mr. Freeman (page 79), viz.: "Mr. Smith used substantially the same data as that from which Mr. Weston satisfied himself that the Darcy formula was satisfactory, and came to a different conclusion" * * * * — I would call attention to the diagrams from No. 14 to No. 22, inclusive, which express very plainly my reasons for recommending the formulas of Darcy for new cast-iron pipes.

In regard to my experiment with the 1-inch pipe mentioned by Mr. Freeman (page 80), I will remark: *First*—that the condition of the in-

Digitized by Google

terior surface of the pipe, with the tin bushings in place, was about the same as the interior surface of an ordinary lead pipe, as I have already stated. I came to this conclusion by a careful examination of the different lengths as they were joined together while being laid. *Second*—that there is a possibility that the data given in Table No. 1, relating to this experiment, may be in error to the amount of 1 per cent. *Third*—that the loss of head in the ⅛-inch tap was determined as follows: A short piece of lead pipe was soldered to the tap, before the 1-inch tin-lined pipe was connected, and a loss of head ascertained due to the discharge under the reservoir pressure, by measuring the flow. This loss of head was then compared with a diagram that had previously been constructed from the results of experiments, which covered a wide range of velocities, that had been made with other taps of the same kind connected to other main pipes, and as it agreed almost exactly with the law that had been developed upon the diagram, a loss of head was scaled from the diagram corresponding to a velocity of 6.03 feet per second, which was the velocity in the 1-inch tin-lined pipe during the experiment. The loss of head thus obtained was then added to the head required to generate the velocity and their sum subtracted from the total or reservoir head, the result of which is given in table No. 1 as the loss of head in the 1-inch tin-lined pipe. A considerable difference in judgment, however, one way or the other, in determining the loss of head in the ⅛-inch tap, etc., would not have materially affected the results given in the table, owing to the small proportion that it would bear to the total loss of head, on account of the long length of the 1-inch tin-lined pipe; consequently, I did not consider the method followed in obtaining it of sufficient importance to be described in detail. There was not any measurable loss of head in the 36-inch main to which the ⅛-inch was connected—about 5 000 feet from the reservoir—as the only water moving in it at the time (the main being full and under the reservoir pressure) was the small quantity that was used in making the experiments.

I will answer Mr. Freeman's questions relative to the possible error of my experiments, made with a 6-inch pipe, as follows:

(*a.*) The gauges, which were very reliable, were tested a short time before the experiments were commenced with a mercurial column, and directly before and after the experiments with a large test gauge that was kept especially for the purpose, and which had also been tested a short time before the experiments were commenced with a mercurial column. The dials of the gauges were 7 inches in diameter, graduated to ½ pounds, and read from 0 to 100. The gauges were each observed at least twenty minutes during each experiment.

(*b.*) I do not think there is a question of doubt but that the piezometers were free from air, as great care was taken to blow

Digitized by Google

them off before each experiment, and they were always full of water under pressure.

(*c.*) The piezometric orifices were exactly flush with the interior surface of the pipe, and normal to its axis.

(*d.*) The diameter of the pipe given in Table No. 1 is the "foundry size," as it was not possible under the circumstances, the pipe being under ground, to measure the diameter; but the pipe was exceedingly well made, it having been furnished by one of the best foundries in the country.

(*e.*) It is almost absolutely certain that the pipe did not contain any obstacles like bunches of lead, etc., as each length of pipe was carefully inspected while being laid by a careful and experienced inspector. From what I have seen by examining pieces of other pipe of the same kind, that have been cut out while making repairs or connections and which had been in service about an equal length of time, in Providence, where the experiments were made, I should say that it was probable that the interior surface of the pipe was not quite as smooth as when the pipe was laid, owing to tubercles that may have formed at odd places when the coal-tar coating may have been thin.

(*f.*) The lengths of hose were those that had been gauged.

I have long recognized what Mr. Freeman states, viz.: "that eternal vigilance and everlasting patience are the price of scientific accuracy in this class of work."

In reply to the comments of Mr. Freeman (page 81), which commence thus: "The experiments selected by Mr. Weston in developing his formula for very smooth pipes are in some respects questionable," * * * * and end thus: * * * * "Moreover, in each of these cases of Rennie's and Neville's experiments the co-efficient of influx which has to be guessed at exercises a proportionately larger influence on the total loss of head than for longer pipes"—I will say: *First*—that the data of the experiments, from No. 1 to No. 14, were considered reliable by such an eminent authority as Professor Weisbach, from one of whose works I obtained them, as I have before mentioned, and that I was not able to secure any other information concerning them than what I have given in Table No. 1. *Second*—that Mr. Freeman is in error when he insinuates that I made use of Rennie's experiments in developing my formula, as I only used these experiments for comparison after the formula was constructed, as may be seen in Table No. 1, and upon Diagrams Nos. 1 and 7. *Third*—that the co-efficient of influx was not guessed at that was used with Neville's experiments, as a co-efficient was determined by experiment by Neville especially for these cases, as I have already stated. *Fourth*—that I take exceptions to Mr. Freeman's idea relating to the regimen of flow not being fully established in the pipes

Digitized by Google

that were used by Neville and Rennie in making their experiments. Such might have been the case if the velocities had been very low, but as the velocity of flow in the pipe used by Rennie was not less than 2.35 feet per second, and the velocity of flow in the pipe used by Neville not less than 14.58 feet per second, I do not think there is any question but that the regimen was fully established. As bearing upon the subject, I will state that I have found in experimenting with a ¾-inch pipe, 6 feet in length, that with as low a velocity as 0.90 feet per second, the outlet end of the pipe was flowing full and the water running clear.

In answer to Mr. Freeman's remarks (page 81), which commence thus: "For exact scientific purposes it seems hardly fair to class on the ground of smoothness experiments on new wrought-iron lap-welded gas pipe, ¾ inch and 1 inch in diameter, along with the pipes of glass and lead," * * * * and end thus, * * * * "Give a much greater friction loss than a smooth lead pipe, and cannot properly be included"—I wish to say: *First*—what I have previously stated on page 49, viz.: That the experiments that were made with the new wrought-iron gas pipe, mentioned by Mr. Freeman, were not given the same weight in the investigations as those that were known to have been made with pipes having very smooth interior sides, but were more especially used for the purpose of substantiating the laws and results obtained with the latter. *Second*—that I did not make use of Darcy's experiments which had been made with a new 1-inch wrought-iron pipe in the same manner that I did those of Smith, for the reason that I was well assured that wrought-iron pipes manufactured in France at the time Darcy made his experiments were not as well made, nor did they have nearly as smooth interior sides, as the gas pipe manufactured at the present time. *Third*—that I did not use in determining my general formula for pipes having very smooth interior sides, as I have stated on page 49, the experiments of Darcy that were made with riveted sheet iron pipe, 7.40 inch and 11.69 inch in diameter.

In reply to the statements of Mr. Freeman (page 81), which commence thus: "It is hardly following the true scientific method, but is arguing in a circle," * * * * and end thus, * * * * "Was apparently merely the nominal or commercial diameter"—I will say: *First*—that as the nature of the pipe used in making the two experiments, Nos. 58 and 237, was questionable, I gave myself the benefit of the doubt, knowing that these two experiments would not appreciably influence the results which I should obtain, owing to the large number of other experiments that were used, which were made with pipes known to have very smooth interior sides. *Second*—that I have stated on page 49 that the data obtained from the experiments from No. 124 to No. 139 were not given the same weight in the investigations as the data derived from experiments that were made with pipes having very smooth interior sides.

Digitized by Google

I will say in reply to the following remarks of Mr. Freeman (page 81): "It may be of interest to note that the experiments of Leslie on 2½-inch lead pipe (Nos. 250 to 269), and of Provis on 1½-inch lead pipe (Nos. 175 to 211), though referred to by Mr. Smith, are by him excluded from his results as unreliable"—*First*—that I think I was justified in using the experiments of Leslie and Provis as I did, more especially the former, as I made particular inquiries of Mr. Leslie in regard to them. *Second*—that if the descriptions that I have given are not sufficiently convincing as to the value of these two sets of experiments, I would suggest to those who are specially interested in the subject that they examine the original papers that were written by these two experimenters.

In reply to this question of Mr. Freeman (page 82): * * * "Did the experimenter clearly understand the disturbing influence of air bubbles, and were they driven out and excluded with positive certainty?"—I will remark that the impression that I have derived by reading Darcy's description of his experiments is, that he decidedly knew what he was about.

To the following statement of Mr. Freeman (page 82): "As we thus carefully examine the quality of the data we see how unsatisfactory much of it is as a foundation for a new formula for very smooth pipes, and how very few are the really reliable experiments"—I will reply by saying that I think Mr. Freeman is decidedly wrong when he makes the assertion that very few of the experiments upon which I have based my new formula for very smooth pipes are really reliable, as I maintain that the reliability of the greater part of them should not be questioned, and I think a careful examination of my paper will prove that I am right in this respect, even if I have failed to show that the majority of Mr. Freeman's exceptions to my work, that he has mentioned in detail in his discussion and which I have previously replied to, are unwarranted.

I do not consider Mr. Freeman's comparison (page 82) of surveying a field with the making of hydraulic experiments, a fair one, and I am somewhat surprised after his recent experience in making extensive experiments with fire hose and nozzles,* that he should suggest such a comparison; and I can assure Mr. Freeman, from the results of my own experience, that if I could have made experiments relating to the flow of water in pipes as easily and with as little expense as I could have surveyed a field, I should not have given him or any other member of the Society an opportunity to criticise any experiments other than my own.

I will conclude by remarking, as Mr. Freeman has referred to tables relating to the flow of water in pipes (page 83), that I have appended to my paper in convenient form for platting, a short table that has been

* *Transactions*, Vol. XXI, No. 426, November, 1889; "Experiments Relating to Hydraulics of Fire Streams," by John R. Freeman.

Digitized by Google

calculated from my new formula for the flow of water in very smooth pipes. And that a table for the flow of water in new cast-iron pipes that was computed under my direction from Darcy's formulas, which is similar to the table mentioned by Mr. Freeman, though more complete, may be found in the appendix to the Report of the City Engineer of Providence, for the year 1888. I would also call attention to two other convenient tables relating to the same subject, one of which is calculated from Weisbach's formula, and is included in the discussion of a paper by James Leslie in the "Excerpt Minutes of Proceedings of the Institution of Civil Engineers, Vol. XIV, Session 1854–55," and the other is appended to a "Rudimentary Treatise on Civil Engineering, by Henry Law, C. E.," that was published in London, by John Weale.

Digitized by Google

LOUISVILLE CEMENT.

The undersigned is General Agent for the following Standard Brands of Louisville Cement:

FALLS MILLS (J. Hulme Brand),
BLACK DIAMOND MILLS (River),
SPEED MILLS,
FALLS CITY MILLS,
QUEEN CITY MILLS,
ACORN MILLS,
BLACK DIAMOND MILLS (Railroad),
FERN LEAF MILLS,
EAGLE MILLS.

This Cement has been in general use throughout the West and South since 1830; most of the public works having been constructed with it. Orders for shipment to any part of the country, by rail or water, will receive prompt and careful attention.

Sales for 1888, 1,202,118 Barrels.
Sales for 1889, 1,338,464 Barrels.

WESTERN CEMENT ASSOCIATION,

247 W. Main St., Louisville, Ky.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

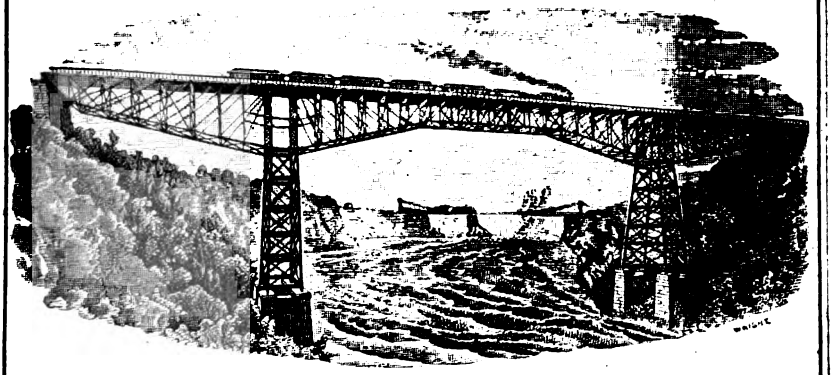
SELECT ADVERTISEMENTS WILL BE RECEIVED AT THE FOLLOWING RATES:

	TWELVE MONTHS.	SIX MONTHS.	THREE MONTHS.	ONE MONTH.
One Page.....	$100.00	$55.00	$30.00	$12.00
One-half Page.....	55.00	30.00	18.00	7.00
One-quarter Page	30.00	18.00	10.00	5.00
One-twelfth "Card".	12.00	.....	.....	....

Address the Secretary of the Society, 127 East 23d Street, New York.

Digitized by Google

UNION BRIDGE CO.



C. MACDONALD, GEO. S. FIELD,

C. S. MAURICE, EDMUND HAYES,

CIVIL ENGINEERS,

And Constructors of Iron and Steel Bridges, Viaducts, Roofs, Elevated Railroads, Marine Piers, etc.

Designs and Estimates will be sent on application to

General Office, WASHINGTON BUILDING,

1 BROADWAY, NEW YORK.

Works: { Athens, Pa. - - Capacity, 15,000 Tons.
Buffalo, N. Y. - - " 15,000 "

Digitized by Google

APR 22 1890

VIII 57

FEBRUARY.

1890.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Library.

CONTENTS.

TRANSACTIONS. VOL. XXII.

No.		Page
432.	Cast-Iron—Strength, Resilience, Tests and Specifications. J. B. JOHNSON....	91
....	Discussion— By GEORGE A. JUST, LOUIS DE COPPET BERG and J. B. JOHNSON.....	121

ILLUSTRATIONS.

XXIX.	Tension Tests of Cast-Iron.....	98
XXX.	Strain Diagrams, Foundry Cast-Iron.....	100
XXXI.	Strain Diagrams, Gun Iron.....	101
.....	Graphical Determination of Moment of Resistance in Tranverse Tests.....	102
XXXII.	Level Micrometer for Measuring Extension	103
XXXIII.	Heisler's Impact Testing Machine.....	111
.....	Machine for Cross-Breaking Tests.....	112
XXXIV.	Diagram showing absence of any relation between Strength and Resilience of Cast-Iron.....	120
XXXV.	Photographs of Specimens Tested.....	124
XXXVI.	Photographs of Lintels Tested.....	128

The Transactions of the Society will be sent to any subscriber at the rate of $10 per year; and to clubs of ten or more, when ordered through the Secretary of an Engineering or Technical Society or Club, who will be responsible for the payment, at 25 per cent. discount.

PUBLISHED AT THE HOUSE OF THE SOCIETY, 127 EAST TWENTY-THIRD STREET, NEW YORK, ON THE SECOND WEDNESDAY OF THE MONTH.

Entered according to Act of Congress, in the year 1890, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Digitized by Google

American Society of Civil Engineers.

OFFICERS FOR 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN, WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Standing Committees.

On FINANCE :—Charles B. Brush, Mendes Cohen, Robert Van Buren.
On LIBRARY :—Alphonse Fteley, Theodore Voorhees, William Ludlow.

Special Committees.

THE PRESIDENT OF THE SOCIETY IS HONORARY MEMBER OF ALL COMMITTEES.

ON UNIFORM STANDARD TIME :—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston, Fred. Brooks.

ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY :—F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, A. V. Abbott.

ON UNIFORM METHODS OF TESTING MATERIALS USED IN METALLIC STRUCTURES, AND ON REQUIREMENTS FOR THESE MATERIALS TO FURTHER IMPROVE THE GRADE OF SUCH STRUCTURES:—G. Bouscaren, William H. Burr, J. G. Dagron, R. W. Hunt, H. G. Morse, P. Roberts, Jr., H. B. Seaman.

MEETINGS.

Regular Meetings, on the first and third Wednesdays of the month, at 20 o'clock, except during July and August.

The rooms of the Society are open from 9 to 17 o'clock each business day, except Saturday, when they are closed at 15 o'clock.

The Library and Conversation Room will also, for the present, be open every *Wednesday Evening*, from 19½ to 22 o'clock. Members are invited to avail themselves of the opportunities afforded on these evenings, both for consultation of books and periodicals, and for conversation.

HOUSE OF THE SOCIETY—127 EAST TWENTY-THIRD STREET, NEW YORK.

Digitized by Google

HARVARD COLLEGE
APR 22 1890
LIBRARY

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

432.
(Vol. XXII.—February, 1890.)

CAST-IRON—STRENGTH, RESILIENCE, TESTS AND SPECIFICATIONS.

By J. B. JOHNSON, M. Am. Soc. C. E.

WITH DISCUSSION.

It is proposed in the following paper to give the results of some experiments at the Washington University Testing Laboratory, on the testing of cast-iron for strength and resilience in tension, compression and cross breaking; to describe the methods and appliances there used; to make a plea for the general adoption of resilience tests of cast-iron; to state a new rule and describe a new machine for cross breaking tests; to explain the high cross breaking modulus of cast-iron; and to propose a set of specifications for engineers to use for cast-iron products.

THE TENSILE STRENGTH OF CAST-IRON.

By strength of cast-iron we usually mean its tensile strength. This can only be found by a direct pull in a testing machine.

The St. Louis cast-irons vary in strength from 17 000 to 36 000 pounds per square inch in direct tension. They will average over 20 000 pounds, and a specification of 25 000 pounds can be readily filled

Digitized by Google

with certainty by any of the better class of foundries. Every shop has its own rules for making its high grade mixtures, and these are generally kept secret. It is safe to say, however, that most of them do not really know the strength of these more expensive mixtures, and very often a cheaper mixture may be a stronger one. I am persuaded that no specification, in this region, should allow a lower limit in strength than 20 000 pounds per square inch in tension, and if the strength is considered an important matter, 25 000 pounds should be stated.

In testing cast-iron in tension, the specimens should first be carefully taken from the same heat as the castings themselves. They should be molded from a pattern, with the usual provisions for intercepting the slag, so as to obtain clean iron and a smooth casting. The size of this specimen should be about 1½ inches in diameter and 14 inches long. This should then be turned down in a lathe, leaving the reduced central portion nearly 1 inch in diameter and some 8 inches long, with rounded angles adjoining the enlarged ends.

This enables surface irregularities to be removed, and flaws, blow-holes or cinder cavities to be discovered, and, if possible, removed by further turning down. It also gives a straight pull in the grips which cannot be obtained on a rough specimen. The turning down also reveals the working qualities of the iron, which is often more important than the strength.

Relative Strength of Skin and Interior.

It is the almost universal opinion that the outer portions of a casting are stronger than the internal fibers, and therefore that cast-iron specimens should be tested in the rough.

Professor Unwin, in his recent exhaustive work on "Testing of Materials of Construction," says "this impression is almost certainly erroneous." The writer knows of no tensile tests going to show that the skin is stronger than the inner fibers, and many tests have been made which prove about equal strength.

There was recently made a series of tests at the Washington University Testing Laboratory especially to determine this relation.

A series of bars were cast from different mixtures of the pattern shown in Fig. 1.

The original bars were 1½-inches round in one part, and 1½-inches by ¾ inch rectangular shape in another. The cylindrical end was then turned

Digitized by Google

down to ¾ inch diameter, thus leaving only the very core of the original form, while the rectangular section was left with the skin on. There were three turned down and enlarged sections, one at each end and one in the middle, for gripping in the machine.

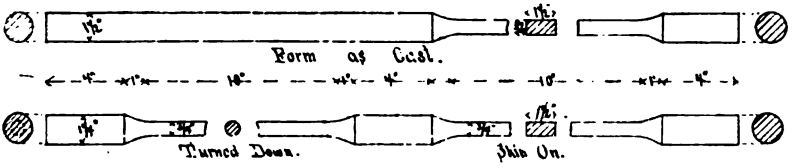


Fig 1

The iron was of a poor quality, so that out of 12 bars tested, only five gave clean fractures on both tests. The results were as follows on these five:

No.	Unit Strength on Turned Down Section.	Unit Strength on Original Section.
2.	19 118	20 387
3.	24 597	15 068
4.	23 896	16 203
6.	23 376	18 817
7.	14 545	12 470
Mean,	21 100	16 600

This shows an average strength for the turned down sections of 4 500 pounds per square inch more than for the corresponding original sections, or an increase of 26 per cent. as a result of turning down from 1¼ inches diameter to ¾ inch diameter.

This result cannot be attributed to any twisting action of the grips, as the gripped surfaces were all turned up true.

I have often observed that the strength of cast-iron is increased by turning off the skin. Thus one specimen 2 inches in diameter broke at 17 820 pounds per square inch, while the same specimen turned down to 1¼ inches broke at 26 000 pounds per square inch. Two other raw specimens broke at an average of 16 100 pounds, and when turned down had a mean strength of 18 900 pounds per square inch.

I conclude, therefore, that the outer parts of a casting have no excess of strength over the inner fibers. Neither must we conclude that they are

Digitized by Google

weaker, as the above tests would appear to show. These tests probably indicate that there are great internal stresses in ordinary castings, resulting from the cooling of the outer shell while the interior portions are still in a melted state. When this core comes to cool, and contract, it is prevented from so doing by the now rigid exterior, and so very severe stresses are caused, the interior fibers being in a state of tension and the exterior portions in compression. When this exterior is turned off, it relieves the stress on the interior fibers, which can now exert their full strength in overcoming an exterior force. If we should bore out the interior, I have no doubt the outer fibers would then show the same increase of strength which the inner fibers now show on the removal of the outer parts.

The Tensile Resilience of Cast-iron.

The resilience of a body is the work it does in resisting distortion. The elastic resilience is the work it may exert in resisting distortion, up to the elastic limit of the material, or up to the point where a part of the distortion becomes permanent. The total resilience of a body is the work it is capable of performing in resisting distortion up to the point of rupture.

Resilience must therefore be measured in foot-pounds, or inch-pounds. In this paper it is always given in inch-pounds. Resilience may also be defined as the amount of energy the body will absorb up to the elastic limit, or up to the point of rupture. A body weighing 10 pounds and falling 10 feet (120 inches) has stored in it in the form of active energy, or *vis viva*, $10 \times 120 = 1\,200$ inch-pounds of energy, or work. This is the work gravity has done upon the body, and if this falls upon a piece of cast-iron, for instance, it must absorb this 1 200 inch-pounds of energy, or else break under the shock.

Since cast-iron usually breaks under some kind of a shock, it follows that the resilience of cast-iron is many times more important than the strength. To find the tensile resilience of cast-iron, it is necessary to measure its extension under stress, and to plot the resulting observations. This curve is called a strain diagram.

Strain Diagrams of Cast-iron.

In Plate XXIX are given many such diagrams. The vertical co-ordinates represent tensile stress in pounds per square inch, and the horizontal co-ordinates stretch per inch in length, or relative stretch.

Digitized by Google

Since 1 inch vertically represents 5 000 pounds per square inch and 1 inch horizontally represents 0.001 inch elongation per inch in length of the specimen, then 1 square inch of the diagram represents 5 000 $\times$ 0.001 = 5 inch-pounds per cubic inch of metal.

In examining the curves shown on this plate one is impressed with the fact that they are curved throughout. They are not made up of a straight line, up to what is called the elastic limit, and then curved, as is the case in all rolled or drawn metals, but the varying relation between stress and strain, or distortion, obtains from the start. Since the modulus of elasticity is the ratio of stress to strain, we see at once that a different ratio would be obtained for every different point of any one of these curves.

The cause of this continuous changing relation is probably due to the initial internal stresses in the material resulting from irregularities in the cooling. If the load be removed at any point, the specimen shows a permanent elongation. On applying the load again the specimen is perfectly elastic up to the former loading, when it again pulls out on the original curving path. This is shown by the dotted lines drawn on the curve of the Watertown Arsenal tension test on gun iron, Plate XXIX. These show that after cast-iron has been loaded up to, say, 20 000 pounds to the square inch, it is then perfectly elastic up to that limit, in this case with a modulus of elasticity of about 23 000 000. When the load was removed at 30 000 pounds per square inch, it showed a permanent set of 0.0032 of its length, and a modulus of elasticity of 18 750 000 up to this limiting stress. On the other hand, if we should have taken its initial rate of distortion on first straining it up to 30 000 pounds per square inch, from this upper part of the curve, we would have obtained a modulus of elasticity of some 910 000.

That is to say, to find the action of cast-iron on its first loading, we should study the full line strain diagrams, but to learn its action on any subsequent loading, we should study the dotted lines drawn to the Watertown Arsenal curve in Plate XXIX. In general, these lines, showing the action of the iron after the first loading, may be taken parallel to a tangent to the curve at the origin, or, when near the ultimate strength, a little more inclined from the vertical.

Although most of these curves have a general resemblance to the parabola, it is evident they cannot be represented by any one parabola, and no one such equation can be written to represent them. The curve

Digitized by Google

marked No. 44 is almost a straight line throughout, and its modulus of elasticity is as low as 8 000 000.

In the curves here shown the modulus of elasticity varies from 8 000 000 to 23 000 000, and therefore it is impossible to assign it any fixed value for the purpose of computing deflections or other distortions of any cast-iron member.

Computing Resilience from Strain Diagrams.

If a vertical ordinate be drawn to the upper extremity of a stress-strain curve, the area included between it, the curve itself, and the horizontal axis represents the work done on the specimen, and absorbed by it before rupture occurred. In other words, this is the resilience of the material. In the curves here shown, the stress is given in pounds to the square inch, while the stretch is given in fractions of one inch per inch in length of the specimen. Thus, if a cubic inch of iron stretches 0.001 inch for a stress of 8 000 pounds, then the mean stress during this stretch has been (in case of No. 44, which has a straight line curve) one-half of 8 000 pounds, or 4 000 pounds. This mean stress, into the stretch, or $4\,000 \times 0.001 = 4$ inch-pounds of work done on this cubic inch of iron. But this is the area of this part of the curve multiplied (for the scales here chosen) by 5. For the altitude of this triangle is 1.6 inch, and the base is 1 inch, giving an area of 0.8 square inches, which being multiplied by 5 gives 4 inch-pounds of work, or energy, or resilience.

To evaluate the areas of the curves, a planimeter is used, and the resilience indicated by each of the curves on the accompanying plate has been so determined. These resilience values are given on the plate for each curve.

The maximum resilience here found is 137.5 inch-pounds per cubic inch in case of No. 76, and the least is 17.3 inch-pounds per cubic inch, in case of the mean curve of eleven tests by Hodgkinson in English cast-iron made many years ago.

The greatest total resilience the writer has ever found for cast-iron is 140 inch-pounds per cubic inch.

The contractions "sil." and "alum." used on the plate, are used to designate silicated and aluminum iron respectively. The silicated iron was treated by the Mullins process, while the aluminum iron was sent from the Michigan Stove Works, at Detroit, and is a stove mixture.

The effect of the silica treatment is shown very strongly by contrast-

Digitized by Google

ing curves 40_1 and 44 with No. 46_2. Nos. 40_1 and 44 are two mixtures used at McCormack's shops, Chicago, and No. 46_2 is said to be the same as No. 40_1, but treated with silica.

The resilience is raised from 46 to 130 inch-pounds per cubic inch, while the strength is raised from 16 000 to 20 800 pounds per square inch. Many other tests showed a similar increase in both strength and resilience, but are not plotted here. Nos. 74, 75 and 76 were given as cheap mixtures at Deerings' works, Chicago, treated with silica. They were such mixtures as could not be used, except for the silica treatment, and yet one of them, No. 76, shows a resilience in excess of the gun iron tested at the Watertown Arsenal.* This particular test of gun iron was chosen because it showed the highest resilience found in any government tests available.†

The Level Micrometer.

Without doubt the most accurate means of measuring a small movement in a vertical plane is a level bubble. It having been the writer's privilege to do a great deal of experimental work with level bubbles, while in the employ of the Government, he conceived the idea of measuring the small extensions of cast-iron specimens and of other metals inside the elastic limits by this device. His testing machine is one of Riehle's "Harvard" type, of 100 000 pounds capacity. The specimen stands vertically. It is necessary to use two bubbles, so as to eliminate the effect of the movements of the machine itself during the test.

The apparatus is shown in Plate XXXII and explains itself. The two collars, *C*, are fastened to the specimen by three pointed steel screws. Each collar carries two armatures symmetrically arranged on opposite sides. The upper armatures carry stirrups suspended from small spurs, while the lower ones carry thumbscrews, *S*, pointed upward. The bubbles, *A* and *B*, are set on as shown, and brought to a horizontal position by the aid of the thumbscrews. Each bubble rests at three points, 2 feet being in the stirrup and the upper point of the thumbscrew, which fits into a conical socket of the bubble frame, making the third.

* A series of twenty-eight tests of identical mixtures, except for silica treatment, fourteen of each class, showed an average increase of tensile strength of 22.6 per cent., while sixteen tests of identical mixtures, eight of each class, showed an average increase of tensile resilience of 90 per cent.

† Taken from Report of Watertown Arsenal Tests, 1884, page 276.

Digitized by Google

When the load comes on and the specimen begins to stretch, each bubble moves toward the stirrup, or suspended end. They thus move in opposite directions, and being symmetrically mounted, one of them measures the sum, and the other the difference of the two movements, namely, the stretch of the specimen and the tilting of the machine. Their half sum therefore records the stretch of the specimen. The angular value of one division of these bubbles has to be determined with great accuracy on a level trier, and a table prepared for each of them giving stretch in inches for one division of bubble movement. To get the stretch for 1 inch in length we must divide this total movement by the length in inches between the clamp screws of the two collars. On Plate XXIX is plotted the stretch per inch in length to stress in pounds per square inch. Each curve, therefore, shows the action of 1 cubic inch of metal.

This apparatus works very well and gives the stretch readily to the nearest ten-thousandth of an inch, but I do not know that it is any better than the better forms of micrometer screw appliances. The chief advantage is, it enables the observer to see the stretch. That is, he sees the bubbles move, and so really sees what the specimen is doing at all stages of the test. Thus it is interesting to note, on a test of cast-iron, for instance, after we pass about two-thirds the breaking load, the specimen continues to stretch under a constant load for some time after the load is on. The apparatus is simple, comparatively inexpensive, and with the aid of a slide rule in making reductions these are not very laborious.

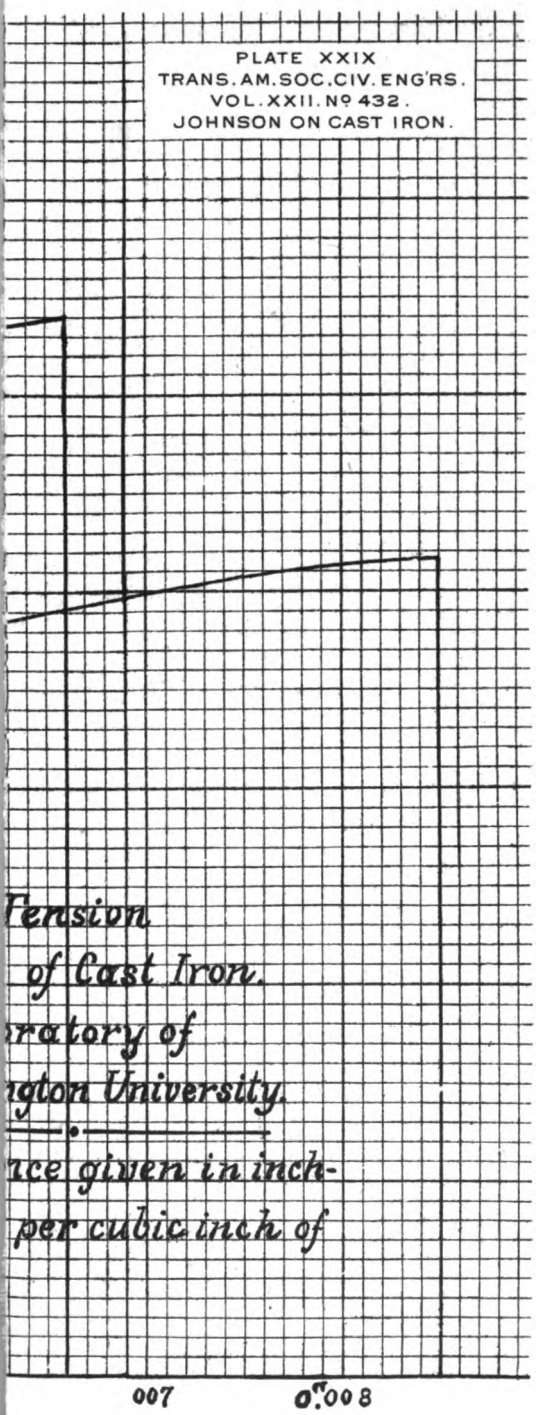
When testing cast-iron the bubbles are removed at about an eighteen or twenty thousand-pound unit stress, and the collars loosened and allowed to rest on the lower cross-head, free from the specimen.

The bubble frames are just 6 inches long between points of support, and one division on the bubble tube is 2 millimeters in length and has an angular value of about ten seconds. A movement of the bubble, therefore, of 1 division is equivalent to a stretch of the specimen of about 0.0003 of an inch.

Cast-Iron in Cross Breaking.

The cross breaking strength of cast-iron can only be found by means of a static cross breaking test. Strength in tension and in cross breaking are not constant functions of each other. In a general way we know

Digitized by Google



Digitized by Google

Digitized by Google

that the computed fiber stress in cast-iron at the time of rupture is about twice the tensile strength. For brittle irons this ratio is less than two, and may even be as low as 1¼, while for tough irons it may be as high as 2½. It also varies with the form of the cross-section. When the mass is concentrated about the neutral axis this ratio is higher, while if it is concentrated in flanges joined by a thin web, this ratio is nearly unity. It is therefore more for a cylindrical test bar than for a rectangular one, and is more on a rectangular bar strained in the direction of a diagonal than when loaded in a direction parallel to one side.

If tensile strength is desired, therefore, a tension test must be made, and if cross breaking strength is sought, only a test in cross breaking will serve.

The reason why the cross breaking modulus is so much greater than the tension modulus, has been sought for by some of our best scientific investigators, but no mathematical explanation has yet proved adequate. In a general way it is due to several errors in the assumptions made in the common theory.

Errors in the Common Theory of Cross Bending.

The more important errors in the common theory are:

First.—That a cross-section which is plane before bending is plane after bending.

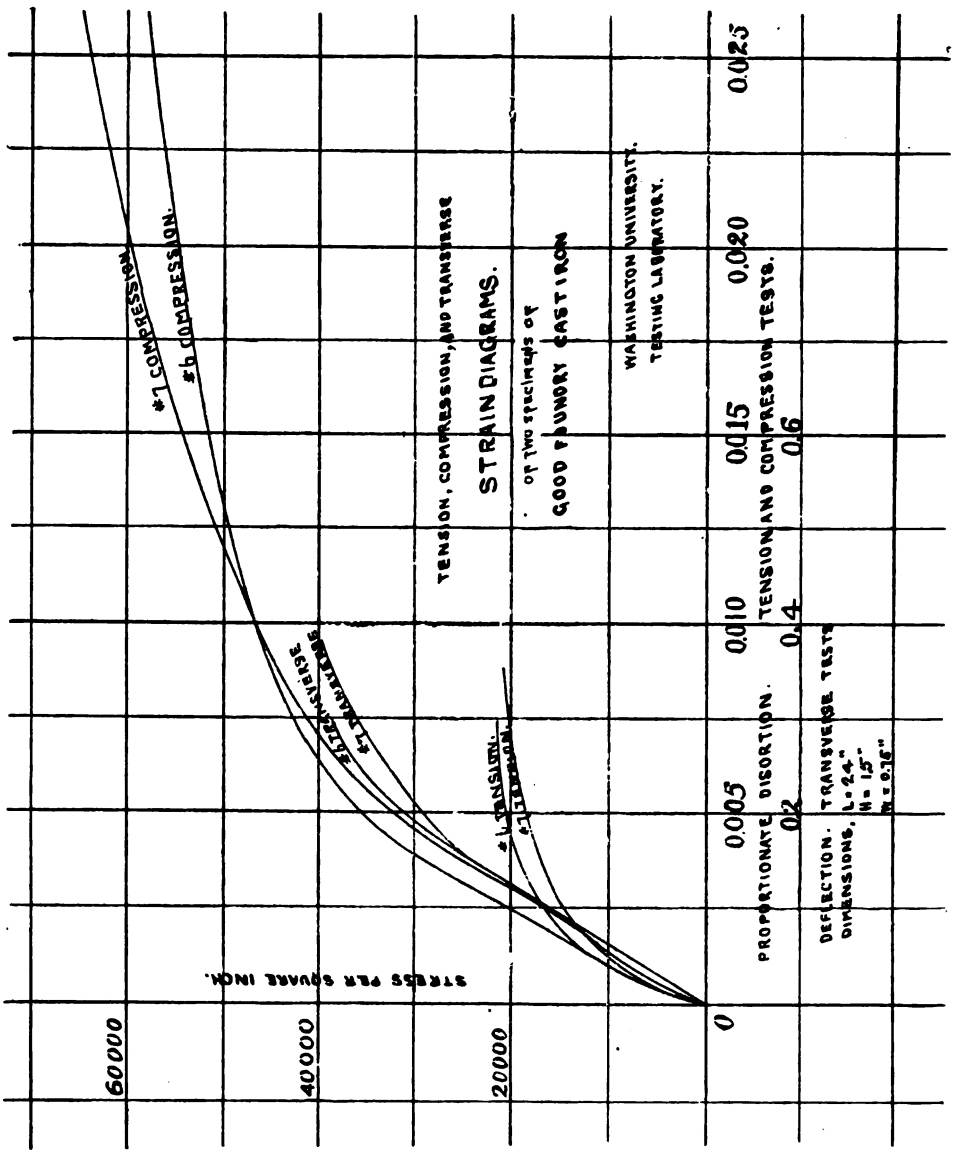
For cast-iron near rupture, and for other metals beyond their elastic limits, this is not true. The writer has made observations on transverse graduations on cast-iron beams under micrometer microscopes of high power, both when free from transverse load and when bent nearly to rupture. A section which is plane before bending is appreciably distorted when near rupture. These investigations are still in progress.

Second.—That the longitudinal stress is uniformly varying across the section.

This is shown to be greatly in error at rupture by the tension and compression curves in Plates XXX and XXXI. That is, we may see from these curves that for equal increments of distortion the increments of stress are very small near rupture, while they are very large near a zero stress. That is, in case of a beam, even though the distortion should vary uniformly across the section, the stress would vary slowly from the tension side inward for some distance, while near the neutral axis it would change rapidly.

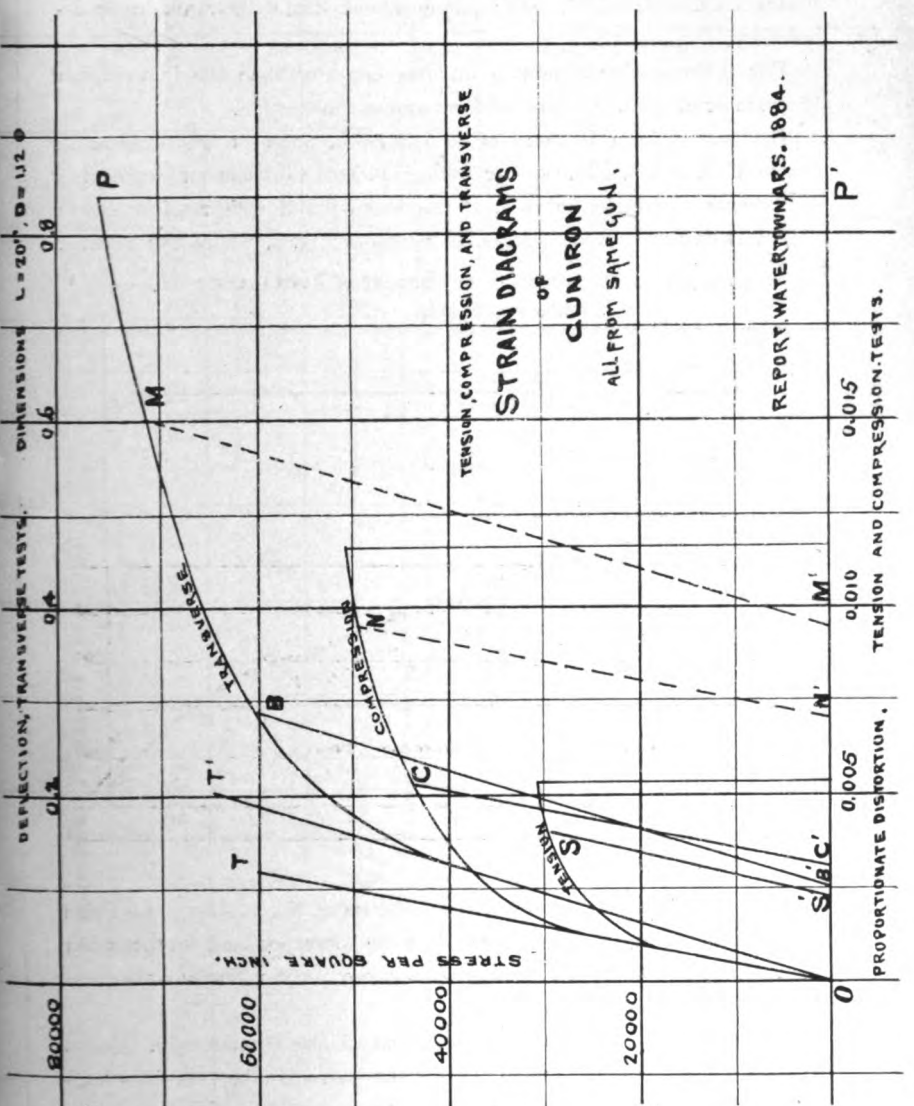
Digitized by Google

PLATE XXX.



Digitized by Google

PLATE XXXI.



Digitized by Google

In fact, if the section remained plane after bending, and up to rupture, then the tension and compression strain diagrams, as shown on Plates XXX and XXXI, may become actual strain diagrams in cross-bending.

For, if the section remains plain after bending, then the increments of strain or of distortion are uniform across the section.

But for uniform increments of distortion, such as are plotted in Plates XXX and XXXI, the curves there plotted show the corresponding increments of stress. So that, if the tension and compression strain diagrams be revolved 90 degrees, as shown in Fig. 2, where the shaded

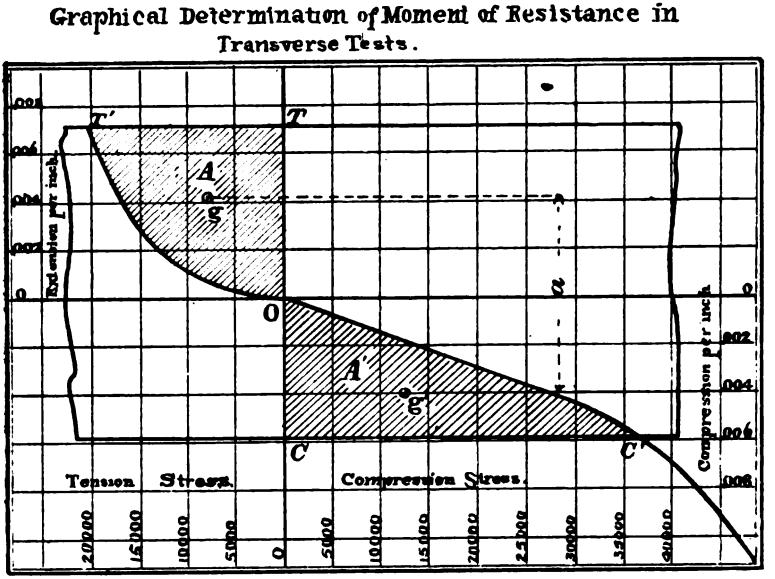


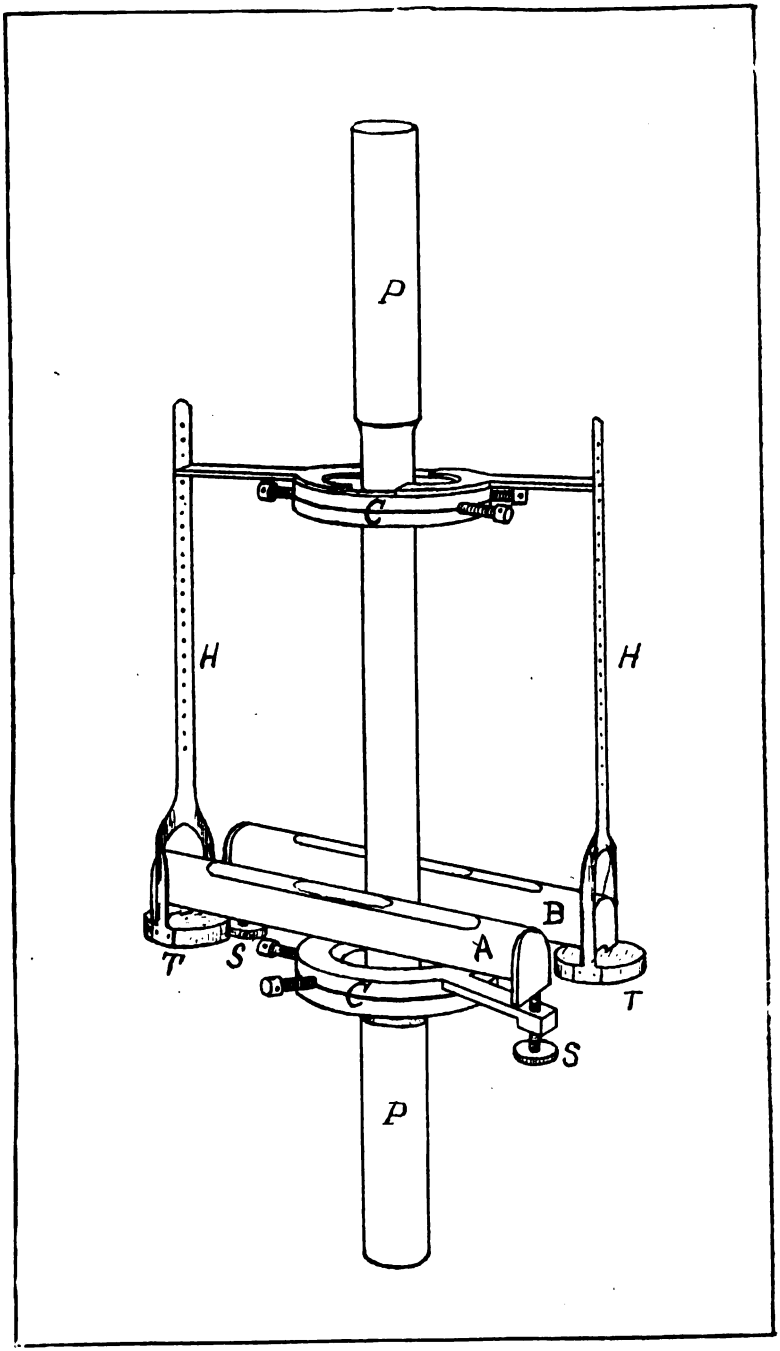
Fig. 2.

portion, $O\,TT^1$, is the tension strain diagram of No. 7, shown in Plate XXX, and the shaded portion, $O\,CC^1$, is the corresponding compression diagram, then horizontal ordinates to these curves represent actual longitudinal stresses in the beam.

Since the beam does actually break first on the tension side, then it is fair to say that the real stress on the extreme fiber on this side at rupture is the breaking strength of the material in direct tension. That is, TT^1, which in this case is about 21 000 pounds, represents the stress on the extreme fiber on the tension side at rupture. Its distor-

Digitized by Google

PLATE XXXII.



Digitized by Google

tion at the breaking point was then about 0.007 of its length. Since the distortion is assumed to vary uniformly across the section, then for distortions of 0.006, 0.004 and 0.002, which are plotted with equal spaces on the figure, the corresponding stresses are about 19 000, 17 000 and 13 000 pounds per square inch, respectively.

If the line TT^1 be taken as the position of the tension side of the beam, then the line through O becomes, to some scale, the position of the neutral axis. It remains to find the position of the extreme fiber on the compressed side on this same diagram.

The compression strain diagram is $O\,C^1$ continued indefinitely. The position of the extreme fiber and the stress on the same cannot be marked off at once.

In any structure or beam, the sum of the longitudinal (or horizontal) stresses or components of stresses, cut by any section, must equal the sum of the outer forces or components in the same direction on one side of the section.

In the case of simple cross-bending the sum of the longitudinal components of the outer forces on one side of any cross-section is zero; therefore, the sum of the longitudinal stresses in the structure or beam itself must be zero. In other words, the sum of the tension stresses must exactly equal the sum of the compressive stresses in every beam subjected to simple cross-bending.

Since the beam actually does break by tension, then the stress on the extreme fiber on the tension side must be the tensile strength of the material, and hence the whole of the tension strain diagram must be used. The area of this diagram (OTT^1, Fig. 2) represents the total stress on the tension side of a rectangular beam 1 inch wide. Since the compressive stress must be equal to it, we must take an equal area from that diagram, by passing an ordinate parallel to the neutral axis, cutting off an area $O\,CC^1$ equal to $O\,TT^1$. When this is found by trial, using a planimeter to measure areas, this horizontal line through C represents the extreme fiber on the compression side of the beam, and the ordinate CC^1 represents the stress on this fiber at rupture.

The moment of resistance is now equal to one of these total stresses (or areas) into the arm of the couple, which is the vertical distance between their centers of gravity. This arm, however, must be the true distance in the specimen itself, and bears the same relation to a as the true height of specimen does to the line $C\,T$ in the figure. That is $C\,T$ represents

Digitized by Google

the height of the beam to some arbitrary scale, and a is given on the drawing to this same scale. The areas must also be evaluated for this scale and that of the stresses.

In the case of specimen No. 7, shown in Fig. 2, the stress area is equivalent to 10 390 pounds, and the arm, reduced, is 0.94 inches. The moment of resistance is, therefore, the product of these two quantities, or 9 760 inch-pounds.

The actual breaking moment was $\frac{1}{4} Wl = \frac{1\,850 \times 24}{4} = 11\,100$ inch-pounds.

The moment of resistance by the ordinary formula is

$$Mo = \tfrac{1}{6} f bh^2 = \frac{21\,200 \times .75 \times 1.53^2}{6} = 6\,200 \text{ inch pounds.}$$

By using the actual stress diagrams, therefore, instead of the assumption of uniformly varying stress, we have accounted for nearly three-fourths of the discrepancy between the formula and the fact.

In several trials of this kind the true stress diagrams have accounted for from one-half to three-fourths of the usual discrepancy.

There is still something to be explained, therefore. The fact that near rupture the section no longer remains plane will explain a part of the remaining discrepancy. The rest of it is probably accounted for by the third erroneous assumption.

Third.—It is, of course, assumed that before bending, the beam is in a perfectly free and unstrained condition. In the case of cast-iron this is never true. The outer portions cool first, and when the interior cools and contracts the now rigid exterior shell is put in compression and the interior in tension. When the beam comes to be bent the initial compression in the extreme fibers have first to be overcome, so that by the time the exterior fiber on the tension side is reduced to a neutral state there is already a considerable load on the beam.

The writer has also investigated this subject by observing section lines across cast-iron beams on both top and bottom sides, and then slicing the beam longitudinally by sawing it into thin strips, and again reading on the same section lines with micrometer microscopes of high power attached to the standard comparator belonging to the University. These observations are still in progress, but enough has been obtained to prove that the outer portions of such a cast-iron beam expand and the inner portions contract when cut into strips.

Digitized by Google

Summary of Errors.

The principal fact left out of account in the ordinary formula is the varying stress across the section, even though the section remained plane. When allowance is made for the section distorting, this further increases the variation of intensity of stress. When we add to these the unknown initial stress of compression in the extreme fibers which must be overcome before any tension is exerted, we probably have quite enough to explain the surprising strength of cast-iron in cross breaking. There is no necessity, in the writer's opinion, for resorting to some occult source of strength arising from the shearing stresses. It is difficult to see how a shearing stress can add to the moment of resistance. Shear is not moment, and cannot be converted into moment, except by acting through an arm. But when the discussion is confined to one cross-section, there can be no arm by which the shear may act. It is simply a case of equilibrium of moment of resistance with the moment of the external forces, and the writer believes he is now able to explain the entire moment of resistance and to prove it by actual test*

When computed from the ordinary formula the stress on the extreme fiber in cast-iron in cross breaking is from one and one-quarter to two and one-half times what it is by direct tension, and this apparent anomaly has given rise to much literature that is more curious than useful.

Transverse Resilience of Beams.

Just as the resilience of a test specimen in tension is given by the area of the tensile strain diagram, so the resilience of a beam is shown by the area of the strain diagram found by plotting loads and deflections. Thus the average load, multiplied into the distance moved (the deflection) is the work done on the beam. This work, or energy, the beam absorbs before breaking.

In Plate XXXI, the strain diagram for the transverse test is plotted for the deflections and the computed stresses on the extreme fiber. Now, 565 pounds load in this instance give just 20 000 pounds computed stress on the extreme fiber, or 1 square inch of this diagram represents $565 \times .02 = 113$ inch-pounds of resilience. The total area of the curve being 12.34 square inches, the total resilience of the beam is $113 \times 12.34 = 1394$ inch-pounds. The volume of this beam was 20 cubic

* This will form the subject of a paper he now has under preparation.

Digitized by Google

inches, hence the unit transverse resilience for this specimen of gun iron is 69.7 inch-pounds per cubic inch, or 258 inch-pounds per pound of iron. This is 56 per cent. of the tension resilience and is an excessively large result. It is five times as great as any transverse resilience the writer has ever obtained, although the tension resilience of this same iron has been exceeded in the writer's laboratory.

In Plate XXX are given the curves of tension, compression and cross breaking tests of some silicated iron specimens cast expressly for experimental purposes, all from the same run. The specimens were 1½-inch square bars, 26 inches long. They were dressed down to ¾ inch in width and broken transversely with the full 1½-inch thickness. One end of each of the broken specimens was cut into two ¾-inch square bars, turned down, and broken in tension. One end of No. 6 and one of No. 7 was then dressed plane on the ends, turned down to 0.73 in diameter and 3 inches long, and broken in compression, giving rise to the compression diagrams.

The tension curves are put in by exterpolation beyond the 18 000 pound line, but the compression diagrams were observed up to the 60 000 pound line. The cross breaking curves were observed up to rupture.

The strength and resilience of this iron in tension, compression and cross breaking are given in the following table, the latter in inch-pounds per cubic inch.

No. of Specimens.	Tension (Mean of Two Tests).		Compression.*		Cross Breaking.	
	Strength.	Resilience.	Strength.	Resilience.	Strength.	Resilience.
6	21 725	118.5	67 000	30 000	38 400	9.8
7	21 100	138.5	76 000	40 000	38 000	12.7

Here the cross breaking resilience is but 8.3 and 9.2 per cent. respectively of the tension resilience, instead of 56 per cent. as was the case of the gun metal test. This is the ordinary ratio, or in general the cross breaking resilience will be found to be about $\frac{1}{10}$th or $\frac{1}{12}$th of the tension resilience, whether found for the pound or the cubic inch unit.

* The specimens bent in a reversed curve, were shortened ¼ inch, but did not crush.

Digitized by Google

Removal of Loads.

In all three tests plotted in Plate XXXI, the load was frequently removed and the permanent set noted. Thus in the tension test, the load being removed at 30 000 pounds, the specimen recovered a part of its distortion, returning on the path SS^1, and leaving a permanent set of OS^1.

In the compressive test, the load being removed at C, the specimen recovered on the line CC^1 with a permanent set, OC^1.

In the cross breaking test the load being removed at B, the specimen recovered its deflection along the line BB^1, with a permanent set of OB^1.

These paths of recovery are in every case approximately parallel to the tangents to the corresponding curves at the origin. This is also shown on Plate XXIX.

The dotted lines on Plate XXXI indicate paths on which the specimen would have recovered, if the loads had been removed at these points.

The elastic resilience, that is, the resilience within the elastic limit, for a uniform bar in tension is* $R = \frac{f^2}{2E}$(volume).

In cross breaking, for a uniform bar loaded in the middle and supported at the ends, the elastic resilience is $R = \frac{f^2}{18E}$ (volume) for rectangular bar, and $R = \frac{f^2}{30E}$ (volume) for round bar, where f is stress per square inch on extreme fiber, and E is modulus of elasticity.

That is, the theoretic elastic tension resilience is 9 times that in cross breaking for a rectangular bar, and 15 times that in cross breaking for a round bar, for the same computed extreme fiber stress.

Furthermore, we see that in every case the resilience varies with the volume of the bar, and is independent of its dimensions. That is, the cross-breaking resilience of a rectangular bar containing 50 cubic inches of iron is the same, whether the bar be 1 inch square and 50 inches long, or 2 inches square and 12½ inches long, or 1 inch wide and 2 inches thick and 25 inches long, or 4 inches wide and ½ inch thick and 25 inches long.

If this iron had an ultimate cross breaking resilience of 10 inch-pounds per cubic inch, then the total resilience of the bar would be 00 inch-pounds. Now a body weighing 50 pounds, falling 10 inches,

* See Rankine's Applied Mechanics or his Civil Engineering for the following equations.

Digitized by Google

or one weighing 10 pounds and falling 50 inches, has stored in it 500 inch-pounds of energy. If it fell on any one of these four bars, at its middle point, and no energy be spent in compressing the points of contact, it would just break the bar, since the resilience of the bar is just equal to the energy of the blow. For resisting such a blow, therefore, all these bars have an equal strength, whereas, for a static load, the 2-inch square bar is thirty-two times as strong as the bar 1 inch square, the formula for strength for a load in the middle being

$$W = \frac{f\, b\, h^2}{l}\, \frac{3}{2}.$$

This apparent paradox is explained by the fact that the deflection of the 1-inch square bar, 50 inches long, would be just thirty-two times that of the 2-inch square bar 12½ inches long, and since the resistance to shock varies as the product of the strength and the deflection, we see that these two products would be the same.

The fact that strength to resist shock depends only on volume and not on particular dimensions, though of great practical significance, does not seem to be generally known.

Impact Tests to Determine Transverse Resilience.

A common method of testing cast-iron to determine its ability to resist shocks or blows, is by means of some kind of impact, usually a falling body. In Plate XXXIII is shown a machine for this purpose, invented I believe by Mr. Heisler, formerly of the Baldwin Locomotive Works, but now a student in Cornell University. The test bar is held in place by thumb-screws on knife edges, and a third knife-edged center bar rests against the middle of the test bar. A heavy weight is suspended as a pendulum, and is raised to given vertical heights as shown by a proper graduation on the arc. In falling it obtains an energy or *vis viva*, equal to its weight into the vertical distance fallen through, and this energy is supposed to be wholly absorbed by the bar. The strokes are repeated each time from a greater height, until the bar breaks. The energy of the final stroke is called the resilience of the bar.

The machine here shown is now in use by the Baldwin Locomotive Works, and at Cornell University.

There are three serious objections to all impact tests:

First.—A part of the blow is of necessity spent on the falling body itself, and on the pivots and on the supporting body, and what proportion this is of the whole energy can never be known.

Digitized by Google

Second.—As is clearly shown in Plate XXXI, which, however, is an extreme case, each blow, near the limiting strength, uses up, or destroys a part of the original resilience of the bar, so that all the last blow shows is what amount of resilience was left in the bar after the previous blows have been given. Thus, suppose a blow, sufficient to strain the material up to *B*, Plate XXXI, to have been given. It has caused a permanent set in the bar of $O\ B^1$, and has permanently taken out of the bar an amount of resilience indicated by the area $O\ B\ B^1$. Suppose the next blow strained it up to *M*. Then it recovers on $M\ M^1$, leaving the permanent set $O\ M^1$, and we have now destroyed that part of its total resilience represented by $O\ M\ M^1$, which is more than half the original amount in the iron. If the next blow breaks it, it need only have an energy equal to the area, $M\ P\ P^1\ M^1$, and this is called the resilience of the iron; whereas, if a blow of double this amount had been delivered at first, it would not have broken it. The more the material submits to permanent distortion, the greater the error from the drop test, unless the blow could be exactly graduated to the specimen, so as to break it, and no more than that, on the first blow. But this involves a previous knowledge of the very thing sought.

We must conclude, therefore, that impact tests on cast-iron cannot be relied on to give any absolute information concerning the original strength or resilience of cast-iron, or of any other material not perfectly elastic up to the point of rupture.

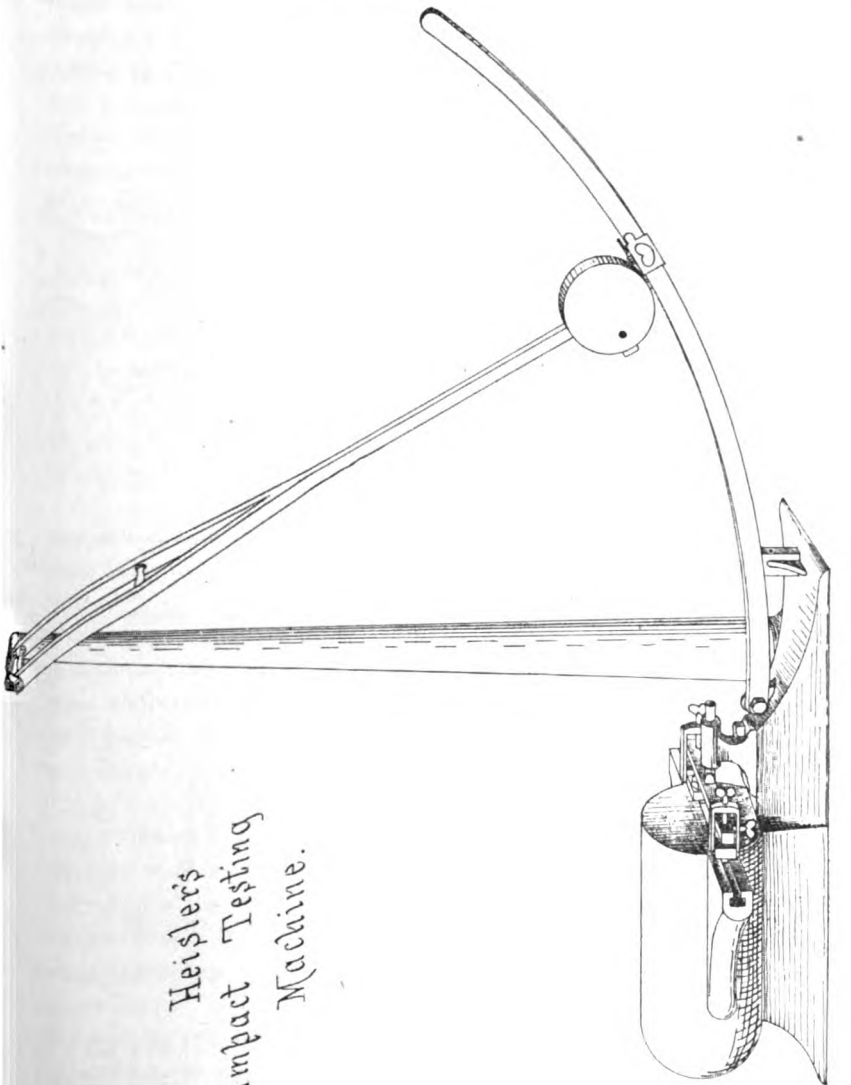
To find the cross-breaking resilience we must make a static test in cross breaking, measure the deflections, plot the resulting strain diagram, measure the area, and compute it per unit volume. This then becomes an absolute measure of the ability of the material in its original state to resist a transverse blow or shock.

For commercial purposes, however, the area of the diagram in cross breaking may be taken as a triangle, when its area would be one-half the breaking load into its maximum deflection. This will always give too small a result, but the error will not often be more than 10 per cent., and since it is always in the same direction, for comparative purposes it will serve very well.

Third.—An impact test gives no information whatever of the strength of the material to resist a static load. It gives some erroneous information of the product of the strength and the deflection, but it tells nothing of either the strength or the deflection alone. The static test, on the

Digitized by Google

PLATE XXXIII.



Heisler's
Impact Testing
Machine.

Digitized by Google

other hand, gives us both these elements separately and then we can multiply them together and get the absolute measure of the resilience. We will then know wherein its great or small resilience lies, whether in its strength or in its ability to distort, and can judge the metal accordingly.

It must be admitted, therefore, that the static test should always be applied when possible.

Machine for Cross Breaking Tests.

Since the existing cross breaking testing machines have no ready means for measuring the deflection of the specimen, and moreover since they cost about $100 each, the writer set about designing a machine which should be at once cheap and adapted to the requirements of a cross breaking test.

The result of his labors is shown in Fig. 3. The weighing is done on an ordinary pair of platform scales. Since the weighing part of a machine is the most expensive part, it was thought that by utilizing the ordinary platform scales, the cost would be greatly reduced. The device here shown is therefore but an attachment to such scales.

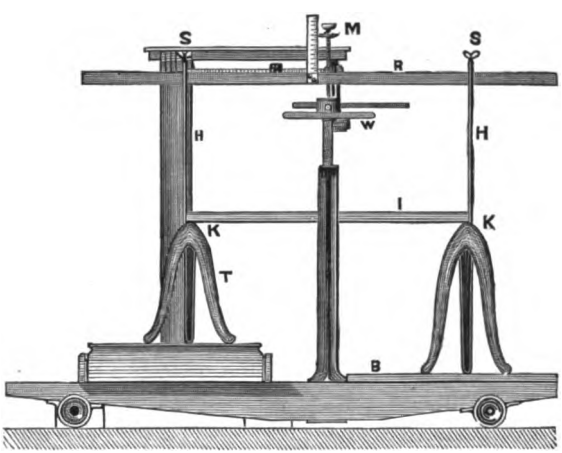


Fig. 3.

The test bar rests on two movable tripods, one of which rests on the scales and the other on the bottom frame of the apparatus. The load is put on by the hand wheel W or by a lever through its capstan head. A reference beam R is held by the uprights H which are rigidly

Digitized by Google

attached to the tripod heads. This reference beam carries a micrometer screw and scale, by means of which the depression of the head of the power screw with reference to the bearing knife edges is accurately measured to the nearest thousandth of an inch. This movement is the deflection of the beam.

Since the ordinary platform scales can weigh loads from 1 000 to 2 000 pounds, it is evident that by this attachment a test load can be applied of twice this amount. But a load of from 2 000 to 4 000 pounds is quite sufficient for any test bar of cast-iron. When not in use the attachment is run off out of the way.*

Since the indicated load, by this machine, is just one-half the breaking load, the resilience of the iron in inch-pounds per pound of metal is given by the convenient rule: Multiply the indicated load (or half the breaking load) by the deflection of the bar and divide by its weight, using pounds and inches as units.

By this rule we have only to weigh the bar and then test it. Or weighing after testing would do as well. But we pay no attention to dimensions.

It is impossible to obtain cast-iron specimens of exact sizes, and hence if the static strength is required, the result has always to be corrected up to what it would have been if the dimensions had been those required in the specifications. This requires a knowledge of formula which the average mechanic does not possess. But here is a rule and a machine which anybody can manipulate, which requires no knowledge of mechanics, costs little, and yet which tells far more about the character of the iron than any test for strength alone, whether in tension or cross breaking.

Since cast-iron usually fails from shock, a resilience test must be applied, and the only way this can be scientifically applied is by means of a static test with measured distortion. The tension test, with measured extensions is good, but it requires an expensive machine and an engineer to run it. But since it is usually the cross breaking strength which is of most value in the product, a test in cross breaking is the proper thing.

By means of this new rule, and with the aid of this simple machine every foundryman could test his own mixtures, and the engineer or his inspector could make his tests for every run of metal at the works.

* These machines are now manufactured in St. Louis, under the direction of the writer, and can be had at a reasonable cost.

Digitized by Google

It has been more or less customary to specify a certain load and also a certain deflection for a given sized specimen of cast-iron, but the loads and deflections have never been combined, so far as the writer is aware, into one result. Thus, if the load was greater than required, but the deflection less, it would be a hardship to the foundryman to have the iron rejected. We now see that it is the product of these two things which is significant, and although one is just as important as the other, yet they cannot be estimated separately. They must be multiplied together.

Values of Tension and Cross Breaking Resilience.

Before the resilience tests of cast-iron can become common, there must be standards established which may be introduced into specifications. Since there have been so few tests made, wherein the deflections have been carefully observed in cross breaking tests, or the elongations in tension tests, it is impossible now to state what such standard requirements should be.

The writer has, however, made some fifty tests of cast-iron in tension, measuring the elongations and computing the tension resilience, and also some 65 tests in cross breaking, measuring the deflections and computing the cross breaking resilience.

Since tension tests are made on turned up specimens of small size it is more convenient to compute the tension resilience in inch-pounds per cubic inch. But cross breaking tests are larger, are irregular in size, and these are more easily weighed. The resilience of cross breaking tests can therefore more readily be computed in inch-pounds per pound of iron.

Values of Tension Resilience.

The writer's results in tension resilience of cast-iron have run from 30 to 140 inch-pounds per cubic inch. The highest tension resilience of gun metal found in the Watertown Arsenal Reports, was 126 inch-pounds per cubic inch, and the lowest found was in Hodgkinson's experiments, or 15 inch-pounds per cubic inch.

I should therefore say that the cheaper grades of cast-iron, such as is used in water-pipe, should be required to have a tension resilience of at least 40 inch-pounds per cubic inch.

For general foundry purposes the tension resilience should be as much as 60 inch-pounds per cubic inch.

Digitized by Google

For foundation parts of machines, engines, pumps, etc., subject to shock, the resilience should not be less than 80 inch-pounds per cubic inch.

For car wheels and all movable parts of machines subject to great shocks, the resilience should not be less than 100 inch-pounds per cubic inch.

All these values to be determined by observing the elongation under a tension test, plotting the resulting strain diagram, as in Plate I, measuring the area, and evaluating it by the scale of the drawing.

Values of Cross Breaking Resilience.

In the following table the values of the cross breaking strength and resilience are given with all the data of the test. The resilience is computed by the rule given above, viz.: $\frac{\frac{1}{2} \text{ breaking load} \times \text{deflection}}{\text{weight of beam}}$.

It will be seen the cross breaking resilience varies from 17 to 51 inch-pounds per pound of iron.

The average resilience for the 40 water-pipe specimens gives 24.3 inch-pounds per pound of iron.

The average of the eleven foundry specimens is 27 inch-pounds per pound of iron.

The average of the four specimens of good stove iron is 44 inch-pounds per pound of iron.

The average of the four specimens of poor stove iron is 21 inch-pounds per pound of iron.

The average of the two specimens, aluminium iron is 49, of the two specimens of silicated iron 45.5, and of the three specimens of Rankine and Fitch 28 inch-pounds per pound of iron.

It thus appears that the cross breaking resilience is more constant than the tension resilience, but still varies in these tests by 300 per cent.

From these experiments I would say that cross breaking tests should show a result, when

$$\frac{\frac{1}{2} \text{ breaking load} \times \text{deflection}}{\text{weight of specimen}}$$

is taken, of

Not less than 20 for water pipe.
" " 30 for foundry castings.
" " 40 for machinery work.
" " 45 for car wheels and movable parts of machines.
" " 50 for stoves and agricultural machinery.

Digitized by Google

WASHINGTON UNIVERSITY TESTING LABORATORY. TESTS OF CAST-IRON IN CROSS BREAKING.

No. of Test.	Dimensions in Inches.			Weight in Pounds.	Breaking Load in Pounds.	Maximum Deflection in inches.	f = Computed stress on extreme fiber. $= \frac{3\,w\,l}{2\,b\,h^2}$	Cross Breaking Resilience in inch-pounds, per pound of iron. $= \frac{\frac{1}{2}\text{ breaking load} \times \text{deflection}}{\text{Weight.}}$	Remarks.
	Width.	Thick-ness.	Length.						
340	2.87	.50	18	6.75	950	.27	35 600	19	Water Pipe.
341	2.78	.50	18	6.75	900	.30	33 800	20	"
352	3.08	.55	18	7.92	1 200	.37	34 800	28	"
353	3.02	.49	18	6.98	1 000	.32	37 200	23	"
364	3.05	.57	18	8.16	1 450	.34	39 400	30	"
365	3.00	.52	18	7.31	1 250	.39	41 700	33	"
428	3.00	.47	18	6.61	750	.30	30 600	17	"
457	3.00	.50	18	7.03	400	.38	32 400	24	"
458	3.00	.52	18	7.31	975	.35	32 300	23	"
575	3.00	.55	18	7.74	1 150	.33	34 200	25	"
576	3.00	.53	18	7.45	1 100	.37	35 300	27	"
613	3.00	.51	18	7.17	1 020	.35	35 300	25	"
614	3.00	.49	18	6.89	1 000	.36	37 500	26	"
776	3.00	.54	18	7.59	950	.37	28 800	23	"
841	3.00	.51	18	7.17	1 090	.31	37 700	24	"
843	3.12	.60	18	8 77	1 275	.28	30 600	20	"
853	3.06	.58	18	8.30	1 580	.26	41 400	25	"
860	3 00	.55	18	7.74	1 275	.84	33 200	28	"
904	3.00	.52	18	7.31	1 140	.30	38 000	23	"
949	3.00	.54	18	7.59	1 200	.30	37 000	24	"
951	2.96	.48	18	6.66	900	.35	35 500	24	"
965	3.26	.56	18	8.53	1 770	.20	46 700	21	"
966	3.00	.53	18	7.45	1 200	.28	38 400	23	"
970	3.20	.52	18	7.78	1 480	.20	45 600	19	"
971	3.00	.63	18	8.86	2 000	.23	45 300	26	"
1 006	3.00	.50	18	7.03	1 000	.33	36 000	23	"
1 007	3.10	.56	18	8.16	1 390	.30	38 600	26	"
1 008	3.05	.54	18	7.74	1 200	.30	36 400	23	"
1 020	3.00	.55	18	7.74	1 250	.33	37 200	27	"
1 021	3 00	.55	18	7.74	1 200	.27	35 700	21	"
1 026	3.03	.61	17.5	8.72	1 275	.33	29 500	24	"
1 027	3.02	.62	17.5	8.77	1 230	.24	27 900	17	"
1 039	3.02	.52	18	7.36	1 070	.35	35 700	25	"
1 040	3.02	.52	18	7.36	1 075	.36	35 800	26	"
1 087	3.03	.57	18	8.11	1 130	.31	31 300	22	"
1 088	3.00	.53	18	7.45	1 220	.43	38 700	35	"
1 139	3.00	.53	18	7.45	1 200	.31	38 500	25	"
1 140	3.00	.49	18	6.89	1 000	.30	37 500	22	"
1 141	2.85	.50	18	6.66	975	.32	37 130	23	"
1 142	2.95	.50	18	6.94	1 120	.43	40 270	35	"
732	3.00	.74	25	14.45	1 650	.47	37 700	27	Foundry.
733	3.00	.73	25	14.26	1 650	.43	38 200	25	"
1 178	2.00	.28	22	3.21	170	.93	35 700	25	"
1 179	1.04	1.05	22	6.26	1 080	.27	31 400	23	"
1 180	2.00	.27	22	3.09	175	1.00	39 500	28	"
1 181	1.04	1.05	22	6.26	1 020	.21	29 400	17	"
1 182	1.00	1.00	22	5.73	1 465	.27	48 350	35	"
1 183	1.04	1.00	22	5.96	1 085	.25	34 400	23	"
1 184	1.03	1.02	22	6.02	1 200	.26	37 000	26	"
6	1.50	.75	24	6.90	1 800	.29	38 400	38	"

Digitized by Google

WASHINGTON UNIVERSITY TESTING LABORATORY. TESTS OF CAST-IRON IN CROSS BREAKING.—(*Continued.*)

No. of Test.	Dimensions in Inches.			Weight in Pounds.	Breaking Load in Pounds.	Maximum Deflection in Inches.	f = Computed Stress in Extreme fiber. $= \frac{3\,w\,l}{2\,b\,h^2}$	Cross Breaking Resilience in inch-pounds, per pound of iron $= \frac{\frac{1}{2}\text{ breaking load} \times \text{deflection}}{\text{Weight.}}$	Remarks.
	Width.	Thick-ness.	Length.						
4	1.50	.75	24	6.90	1 500	.27	32 000	29	Foundry.
1 192	3.00	.08	14	0.93	64	1.28	70 000	44	Western Stove Co.'s good mixture.
1 193	3.00	.14	14	1.63	148	.98	60 000	44	
1 194	3.00	.12	14	1.40	116	1.08	76 000	45	Western Stove Co.'s good mixture.
1 196	3.00	.09	14	1.05	93	1.15	80 400	51	
$1\,074_1$	1.07	1.05	22	6.42	900	.29	26 100	20	Western Stove Co.'s poor mixture.
$1\,074_2$	1.07	1.06	22	6.47	900	.25	25 600	17	
$1\,074_3$	2.00	.28	22	3.21	150	1.09	31 600	25	Western Stove Co.'s poor mixture.
$1\,074_4$	2.00	.27	22	3.09	150	.94	34 000	23	
5	1.04	1.04	17	4.78	1 940	.24	43 600	49	Aluminium iron.
7	1.04	1.11	17	5.09	2 080	.24	44 200	49	" "
7_1	1.50	.75	24	6.94	1 850	.38	38 000	50	Sil Iron (Greene).
9	1.50	.76	24	6.47	1 725	.335	36 ?00	41	" "
1 232	1.50	1.47	24	13.27	3 100	.231	34 440	27	Rankine & Fritch.
1 233	1.50	1.50	24	13.47	3 375	.241	36 000	30	"
1 234	1.53	1.50	24	13.64	3 210	.226	33 460	27	"

RESULTS AVERAGED FOR LIKE SIZES.

40 Tests	3.012	.534	18	7.548	1 171	.319	36 437	24.3	Water pipe.
2 "	3.00	.735	25	14.35	1 650	.45	37 950	26	Foundry.
2 "	2.00	.275	22	3.15	1 725	.965	37 600	26.5	"
5 "	1.03	1.02	22	6.05	1 170	.252	36 110	25	"
2 "	1.50	.75	24	6.90	1 650	.28	35 200	33.5	"
2 "	3.00	.085	14	1.00	78	1.21	75 200	47.5	Western Stove Co.'s good mixture.
2 "	3.00	.13	14	1.51	132	1.03	68 000	44.5	
2 "	1.07	1.055	22	6.445	900	.27	25 850	18.5	Western Stove Co.'s poor mixture.
2 "	2.00	.275	22	3.15	150	.201	32 800	24	
2 "	1.04	1.075	17	4.93	2 010	.24	43 900	49	Aluminium iron.
2 "	1.50	.755	24	6.70	1 787	.357	37 150	45.5	Sil. iron (Greene.)
3 "	1.51	1.49	24	13.46	3 191	.233	34 633	28	Rankine & Fritch.

Digitized by Google

Ratio of Strength to Resilience.

From Plate XXIX it is evident that there is no relation between strength and resilience in tension tests.

The strongest iron there shown, 32 000 pounds per square inch, has a resilience of 85.3 inch-pounds per cubic inch, while another, whose strength is less than 21,000 pounds, has a resilience of 130 inch-pounds per cubic inch.

The sixty-five tests in cross breaking have their strength and resilience in inch-pounds per pound of iron, plotted to each other on Plate XXXIV. The wide dispersion of these points over the sheet show again there is no relation between cross breaking strength and cross breaking resilience.

Conclusion.

The writer believes that in their zeal for studying the wonderful properties of wrought-iron and steel, engineers have, for nearly half a century, almost overlooked the equally useful properties of cast-iron. That although cast-iron almost always fails from some kind of shock or blow, or a repetition of such blows, yet the ability of this material to withstand such shocks has never been determined. That while the greatest refinements have been introduced into the testing of wrought-iron and steel, no one has thought it worth his while to pay any special attention to the testing of cast-iron.

The writer hopes, therefore, that this subject may receive the attention of those members of the society who appreciate the needs of the situation, and that by a general discussion a set of standard specifications may be agreed upon which will at once improve the practice of the foundries, and insure against many of those unfortunate accidents which have given to cast-iron so bad a name.

Engineers have it in their power to greatly improve upon the character of the cast-iron now used, and to educate the iron manufacturers and iron users up to a higher plane of practice. But so long as his customers do not complain the foundryman will use the cheapest mixture he can find on the market that will pass the casual inspection usually given.

It is not uncommon, in writing specifications for cast-iron, for the engineer to specify the mixture. This is a bad practice. It does not insure a definite product, and it often enhances the cost without any

Digitized by Google

corresponding advantage. Since a product is desired, having certain physical properties, the engineer should confine his specifications and tests to the product, leaving the foundryman free to use any mixture he pleases, so long as his iron comes up to the specified standard.

It is well always to specify something in regard to the inner appearance of the iron, since no white, coarsely crystalline, dirty, or cindery iron is fit for ordinary castings. Something in regard to the outer appearance, also, is well, since rough castings, and such as have cold shuts, or blow holes, are undesirable.

A very good test of the toughness of the iron is the facility with which it is indented by a hammer, on a sharp corner. If it is brittle it chips off, if tough, it indents.

So much for general tests, such as can be applied by a casual inspection. If, however, the real strength of the iron, and what is of a great deal more importance, its ability to resist shocks or blows is desired, then further tests are required.

In order to bring out such a discussion, a proposed set of specifications for cast-iron is appended.

Proposed Specifications for Cast-iron.

All castings shall be made of soft gray iron which can be readily planed, turned or drilled. All broken surfaces shall show a dark metallic luster, of uniform color and close-grained texture entirely across the surface. There shall be no blow holes, cinder patches, cold shuts, or mottled spots, either on the surface or on the interior. The surfaces shall be smooth and full, nearly free from sand, and the exterior angles sharp. All cored openings and parts shall be full and true in form without excess or overrun. When struck with a hammer on a sharp corner it shall indent and not break off. All sudden variations of thickness shall be avoided. Defective castings shall not be patched, plugged, or mended by burning or running on new metal.

From every run of metal, or as often as may be required, test bars shall be run. If tension bars are required they shall be 1¼-inch diameter and 15 inches long. If cross breaking tests are to be made the bars should be ¾ inch thick, 2 inches wide and 2 feet long.* Such specimens shall be cast from regular patterns with the same care as is used in other castings as to blow holes, cinder patches, proper peeling, etc., and shall always be cast in duplicate.

* Almost any other dimensions will do as well, but this is a very good size.

Digitized by Google

When tested in tension the iron shall show a strength of 25 000 * pounds per square inch, and it shall show an elongation under test of at least four one-thousandths of its length.†

When tested in cross breaking and its deflection taken, one-half the breaking load in pounds, multiplied by the deflection in inches and divided by the weight of the bar in pounds shall give a result of more than 30.‡

If static strength is required, and the cross breaking test is to be used, say:

The cross breaking load on the bar shall be more than that shown by the formula $W = \frac{24\,000\, b\, h^2}{l}$. §

Where W is the load on the middle of the bar in pounds, and b h and l are the breadth, height, and length of the beam respectivly in inches.

---

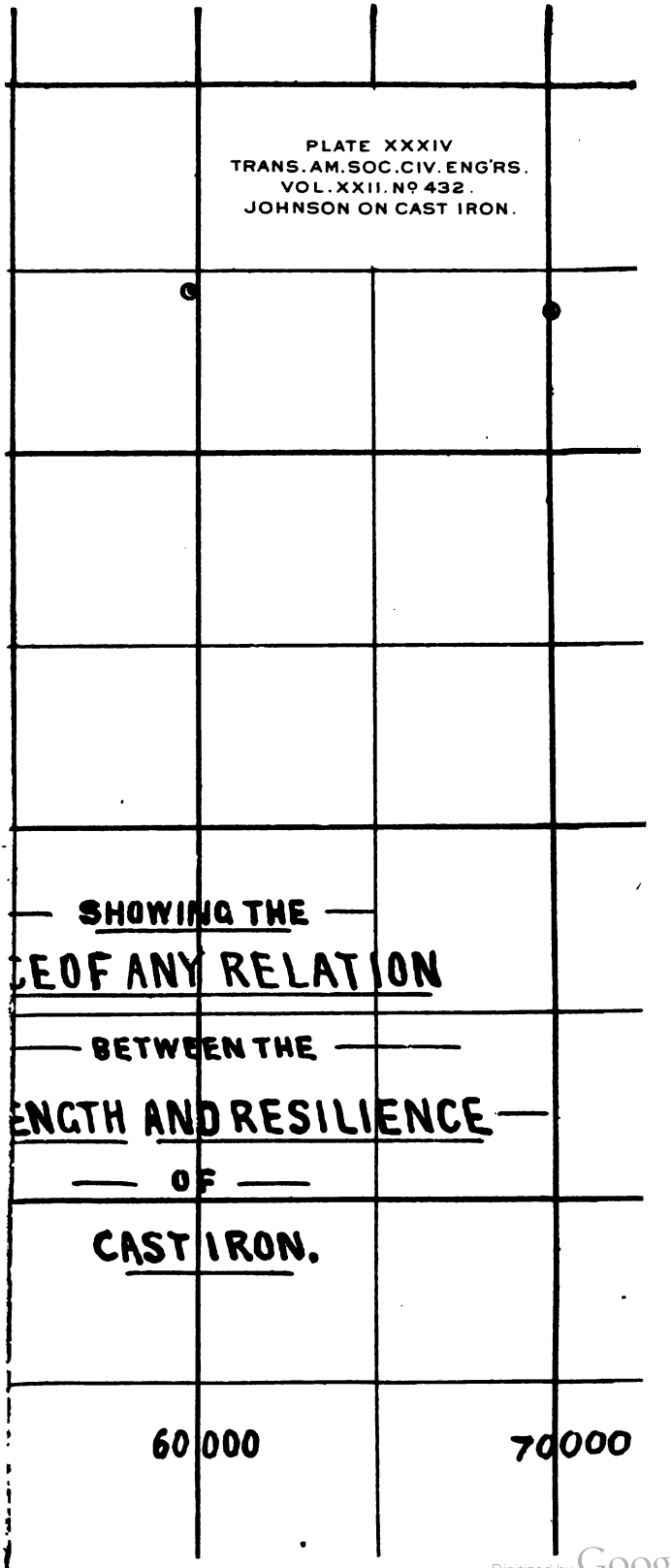
* Put from 18 000 to 28 000, according to requirements.

† See Plate I.

‡ This may be put from 20 to 50 according to requirements. See Table of Cross Breaking Tests.

§ This corresponds to a computed stress in extreme fibers (f) of 36 000 pounds per square inch. If the cross-breaking strength is to be high use 30 000 in place of 24 000. See Table of Cross Breaking Tests.

Digitized by Google



Digitized by Google

Digitized by Google

DISCUSSION.

GEORGE A. JUST, Jun. Am. Soc. C. E.—Designers and manufacturers alike are indebted to Professor Johnson for this record of his experiments, since existing data are, to a great extent, derived from English irons or from high grade American gun irons.

Remembering that his tests were made on ordinary foundry irons, and not on special mixtures designed to give great strength, the feature of the record is that the specimens should have developed a tensile strength from 17 000 to 36 000 pounds per square inch, and his recommendation that specifications should demand from 18 000 to 28 000 pounds. Our experience makes 18 000 pounds a maximum rather than a minimum for the tensile strength of good cast-iron as used in ordinary every day work.

The higher tensile strength which Professor Johnson found is desirable, not because designers now use cast-iron in direct tension, but because of the direct relation existing between the tensile and cross-breaking strength of the material, the latter coming into play continually in machinery castings and in the lintels, column-brackets, etc., of buildings. However, we should like to know if this higher tensile strength was not secured at the expense of the "working qualities" of the material, the importance of which Professor Johnson recognizes when he says the iron should be "readily planed, turned or drilled." We know of iron high in tensile strength, but its hardness makes its working under tools expensive. But if this higher tensile strength is demanded, why not demand a corresponding increase in the cross-breaking strength of the material. Ordinary 18 000 pounds iron will, on a 12-inch span, if 1 inch square in section, just sustain a center load of 2 000 pounds, which by the "common theory" of flexure, corresponds to an extreme fiber stress of 36 000 pounds. The formula proposed:

$$W = \frac{24\,000\,bh^2}{l}$$

demands exactly the same stress. Cooper's and other standard bridge specifications, say a bar one inch square must sustain a center load of 500 pounds on a span of 4 feet 6 inches, and this by the "common theory" corresponds to a stress of 40 500 pounds on the extreme fibers.

It may be well for a specification to say, "There shall be no blow holes, etc., either on the surface or on the interior," but the latter would certainly be difficult to locate, especially in large castings. That "sudden variations in thickness shall be avoided" is a rule to be applied by the designer, rather than the manufacturer, and when it is said that angles must be sharp, we presume this refers to exterior angles or such

Digitized by Google

as fix the outlines of the casting, for all other angles, *i. e.*, those connecting various parts, are always made rounded or with a fillet.

In this connection the results of some tests made in this city for the Jackson Architectural Iron Works, under the direction of the writer, may be of interest.

It was desired to determine any difference in strength which might exist between metal poured from the beginning, middle and end of the same heat. The results failed to indicate such a difference, but the fractures of pieces broken in the same manner generally presented a varied appearance, No. 1 giving the coarsest and No. 3 the finest fracture.

All the test pieces were 1 inch square in section, cast in green sand, from a wood pattern, and poured on their sides, *i. e.*, not on end. They were simply brushed, to clean from sand and the fins removed on an emery wheel. They were moulded on a bench in a "snap flask" so that from 2½ to 3½ inches of sand surrounded the metal when poured. They remained in this condition from ten to fifteen minutes and were then dumped from the flasks.

In charging the furnace, the fire was lighted at say one o'clock, and the charging commenced at two. This was continued until the cupola contained eight tons. The blast was now put on and the charging continued. In this way some twenty tons were run down in about two and a half hours. In firing, shavings and wood were put on the bed of the cupola, followed by 1 500 to 1 800 pounds of hard coal, and then a "draft" with the scrap next to the coal.

A "draft" consisted of 1 000 pounds, and was made up as follows:

300 pounds of Sloss (Southern iron).
300 pounds of No. 2 X (Thomas iron).
200 pounds of Low Moor (Virginia iron).
200 pounds of good stove scrap.

When six "drafts" (about 3 tons) had been fed, 400 pounds of coke were added. The drafts were continued, with coke at intervals, so that about 2 000 pounds of coke were used to 20 tons of metal.

The following average analysis of the pig used was furnished by the manufacturers:

SLOSS OR SOUTHERN IRON.

Phosphorus	0.672
Silicon	4.080
Sulphur	0.025
Manganese	0.239
Combined Carbon	0.271
Graphitic Carbon	3.164
Iron (by difference)	91.549

Digitized by Google

Four samples of No. 2 X Thomas, all from the same cast, gave an average tensile strength of 17 160 pounds per square inch, and

Phosphorus	0.733
Silicon	1.646
Sulphur	0.017
Manganese	0.227

The tests about to be recorded were made on the 150 000 pounds Fairbank's machine in this city. They are mainly of interest, because all the specimens were cast from the same heat, and their strength in tension, compression, cross-breaking and shear are directly comparable. Twelve tests were made in compression, six on 1-inch cubes and six on pieces 1 x 1 x 8 inches long. Six tension tests were made, and six in cross-breaking on a 12-inch span on pieces 1 x 1 x 16 inches long. Six tests were also made on specimens as shown in Fig. 4, and two on full-sized lintels, such as are used in present building practice. Each specimen was marked 1, 2 or 3, showing if the same was poured from the beginning, middle or end of the heat. Since two specimens from each part of the heat were made and tested, a distinctive letter was added. Thus: pieces marked 2*A* and 2*B* were poured from the middle of the heat, were similar and should show similar results. The two lintels were cast separately, but from the same mixture and in the same manner as the smaller specimens.

ONE INCH CUBES.—COMPRESSION.

Mark.	Weight in ozs.	Dimensions in inches before Test.			Area.	Ult. strength in lbs.	Length after Test.	Per cent. of Reduction.	Ult. strength per sq. inch.
		Length.	Width.	Thickness.					
1*A*	4.156	1.015	1.015	1.005	1.020	122 540	.889	12.4	120 137
1*B*	4.156	1.032	1.005	1.003	1.008	116 000	.901	12.7	115 079
2*A*	4.219	1.012	1.004	1.026	1.030	126 290	.883	13.6	122 612
2*B*	4.219	1.016	1.024	1.018	1.042	129 770	.882	13.2	124 539
3*A*	4.250	1.016	1.016	1.016	1.032	125 720	.890	12.4	121 822
3*B*	4.250	1.018	1.020	1.015	1.035	131 220	.875	14.0	126 783

Average ultimate strength per square inch=121 829 pounds.

Average reduction in length=13 per cent.

Average weight per cubic inch=4 ounces.

The specimens were tested between steel dies. After the first test, these were indented, and subsequent specimens did not get a good seat as marks on bearing surfaces of the specimens show. This, however, did not seem to influence the results. The point of failure was marked, the specimens failing by bulging at the sides, and the appearance of fissures on the same. The pressure on "1*A*" was released at 110 380 pounds, with no sign of failure.

Digitized by Google

COMPRESSION—1 x 1 x 8 INCHES LONG—PLANED ENDS.

Mark.	Dimensions before Test—in inches.	Area.	Clicking Sounds.	Yielded Rapidly at	Ult. Strength per sq. inch.
1*A*	1.020 x 1.020 x 8.001	1.0404	.....	61 630	59 260
1*B*	1.025 x 1.025 x 8.018	1.0506	52 000	55 740	53 065
2*A*	1.015 x 1.040 x 8.008	1.0556	40 000 ?	55 000	52 083
2*B*	1.038 x 1.040 x 8.007	1.0795	56 000	60 890	56 379
3*A*	1.040 x 1.040 x 8.008	1.0816	50 200	53 660	49 639
3*B*	1.001 x 1.038 x 8·006	1.0390	48 000	55 660	53 571

Average ultimate strength per square inch = 54 000 pounds.

Average length over radius of gyration = 27.

All failed by flexure, the specimens taking the form of a reversed curve, the metal tearing apart on the convex sides as shown on accompanying photograph, Plate XXXV. This tearing was always indicated by two clicking sounds made in quick succession. If we substitute for P in the formula:

$$P = \frac{f s}{1 + a \frac{l^2}{r^2}}$$

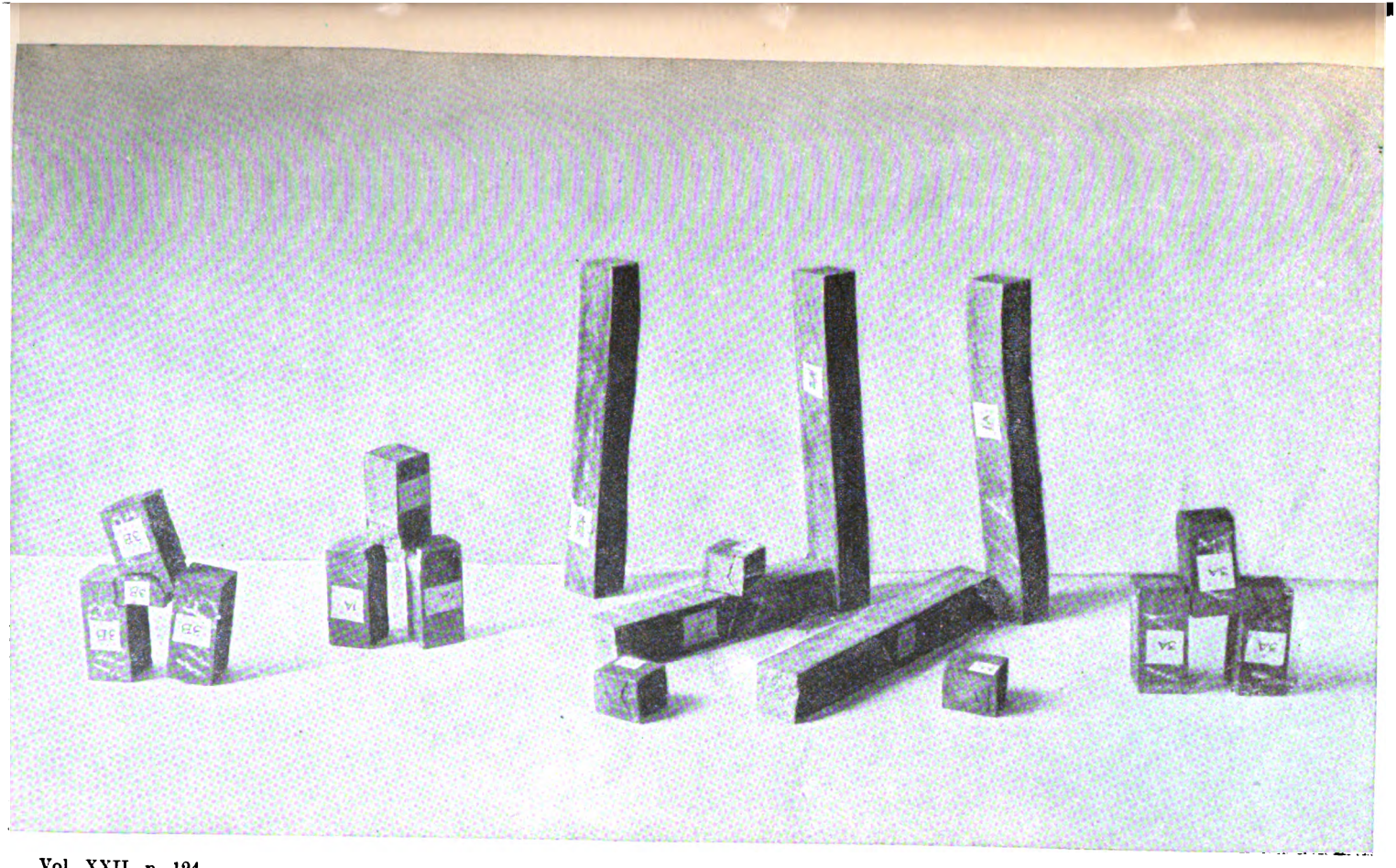
the average ultimate strength found by test, we find $f = 110\,570$ and $a = \frac{1}{714}$. Rankine gives for solid circular sections $f = 80\,000$ and $a = \frac{1}{400}$.

TESTS IN TENSION—SPECIMENS: ONE INCH SQUARE X 16 INCHES LONG.

Mark.	Section in inches.	Area.	Ult. Strength.	Ult. per sq. inch.	Remarks.
1*A*	1.012 x 1.009	1.021	19 380	18 981	broke 2 inches from end.
1*B*	1.008 x 1.016	1.024	18 710	18 271	broke 6 inches from end.
2*A*	1.009 x 1.010	1.019	16 700	16 388	broke 2 inches from end.
2*B*	1.002 x 1.008	1.010	16 530	16 366	broke 3½ inches from end.
3*A*	1.018 x 1.013	1.031	16 180	15 693	broke 4½ inches from end.
3*B*	1.007 x 1.010	1.017	17 070	16 785	broke ½ inches from end.

These tests give an average ultimate strength of 17 081 pounds per square inch. No. 1*A* when tested a second time gave 15 500 pounds, breaking 1½ inches from the other end; when tested a third time it gave 15 400 pounds, breaking 2½ inches from same end. No. 2*A* when first tested gave only 10 480 pounds, breaking 3½ inches from end and developing a flaw covering say 25 per cent. of the section. A second test gave the value shown in the table. No. 2*B* when tested a second time gave 17 820 pounds, breaking 2 inches from first fracture. No. 3*A* developed a

Digitized by Google



Vol. XXII, p. 124.

JUST ON CAST IRON.

PLATE XXXV.

Digitized by Google

Digitized by Google

slight flaw near one face. No. 3*B* when tested a second time gave 14 770 pounds, breaking 3¾ inches from first fracture. When tested a third time it gave 13 870 pounds, breaking 2½ inches from the other end. It might be expected that the first pull would develop the weakest section, but the record shows that the second and third tests generally gave a lower value than the first. The testing machine was provided with large hemispherical heads, intended to eliminate bending and produce a direct pull. Unfortunately, no devices of sufficient delicacy were on hand to take measurements that would permit the calculation of the modulus and the limit of elasticity. To the eye the elongations were inappreciable. Some years ago the writer tested (on a 40 000 pounds Riehlé machine) specimens similar to the above. They were cast by a Trenton, N. J., foundry, the material being intended for the mechanism of a draw-bridge. Three pieces from the same cast gave respectively 16 000, 17 800 and 19 700 pounds per square inch. Another piece from a different cast gave 14 000 on first pull and 10 000 pounds when tested a second time. This specimen broke near the clamps and bending may have taken place. Both fractures were finely granu'ar and without flaws. While nearly all the specimens broke near the clamps it is not believed that the pressure exerted by the latter influenced the results.

TRANSVERSE TESTS ON PIECES 1 INCH SQUARE BY 16 INCHES LONG. CENTER LOAD ON 12-INCH SPAN.

Mark.	Section in inches.	Area.	Deflection at Rupture.	Ultimate Strength in pounds.	Modulus of Rupture.
1*A*	1.010 x 1.018	1.028	....	2 000	34 000
1*B*	1.014 x 1.015	1.029	.120	1 809	30 700
2*A*	1.009 x 1.011	1.020	.100	1 800	31 210
2*B*	1.005 x 1.005	1.010	....	1 800	31 770
3*A*	1.016 x 1.012	1.038	.100	1 850	30 833
3*B*	1.009 x 1.015	1.024	....	1 700	30 000

These tests show an average ultimate strength of 1 825 pounds and an average modulus of rupture of 31 400 pounds. The deflections under load were recorded as follows :

Load in pounds.	1*A*	1*B*	2*A*	2*B*	3*A*	3*B*
500	....	.045	.016	.037	.030	.025
1 000	....	.075	.046	.063	.058	.050
1 200	....	.085	.053	.077	.068	.065
1 500	....	.096	.075	.096	.085	.080
1 700	....	.104	.080	.100	.096	Broke.
1 800	....	.120	.100	....	.100	....
1 850	....	....	....	....	Broke.	....

Digitized by Google

These deflections are not always directly proportional to the load, No. 3*A*, however, approximates to this law. They also indicate the well known fact that the modulus of elasticity in cast-iron decreases rapidly with increase of load. All the loads were applied by means of a block, rounded at the pressing edge by an arc of a circle $\frac{1}{4}$ inches in diameter.

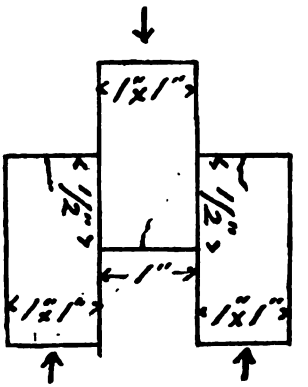


FIG. 4.

SHEAR SPECIMENS AS PER SKETCH, FIG. 4.

Mark.	Section.	Length of Contact in inches.	Clicking Sounds.	Ultimate Strength.	Remarks.
1*A*	1.015 x 1.026	.5275	....	43 830	
1*B*	1.028 x 1.010	.5175	32 000	41 220	
2*A*	1.011 x 1.025	.5300	34 000	47 680	
2*B*	1.090 x 1.036	.5100	....	47 400	Flaw.
3*A*	1.010 x 1.026	.5150	....	41 250	
3*B*	1.005 x 1.035	.5050	....	39 680	Broke into four pieces.

These tests show an average ultimate strength of 43 520 pounds. The clicking sounds were produced by the tearing of the metal on the under side of the piece pressed, and on the upper surfaces of the supporting pieces as indicated in the sketch. The sounds were not noted in each case, but the cracks appear on all the specimens. The tests are termed "shear" tests for want of a better name, but are of interest because they show that the piece pressed acted as a fixed beam which failed by rupture on its under side and at the points of support, and only after this did shearing take place. After failure the first three specimens showed that true shear had taken place along both surfaces of contact. The last three showed only one sheared surface, the other having attached to it an irregular piece broken from one of the

Digitized by Google

supporting pieces. The first three were without flaw, 2*B* had a marked flaw, and Nos. 3*A* and 3*B* a flaw barely visible. The latter broke into four pieces. It would seem, then, that in a column where a part of the load is applied by means of brackets (as is usual in case of house work), failure would take place, by tearing the bracket bodily from the column and not by shear. This would suggest the use of cross-webs within the column at the brackets, when the loads carried by the latter form a large proportion of the total load.

There remains to be recorded the test of two lintels intended to span openings in the walls of buildings, and made in the usual manner.

GENERAL DATA.

LINTEL "A."		LINTEL "B."	
Weight	306 pounds.	Weight	275½ pounds.
Length	6 feet 3½ inches.	Length	7 feet 4 inches.
Width of bed	12 inches.	Width of bed	12 inches.
Center depth	12 inches.	Center depth	8 inches.
Depth at ends	4 inches.	Depth at ends	4 inches.
Average thickness of flange	1⅛ inches.	Average thickness of flange	⅝ inches.
Average thickness of web	⅝ inches.	Average thickness of web	½ inches.
Clear span	5 feet 6 inches.	Clear span	6 feet.
Ultimate strength	98 450 pounds.	Ultimate strength	43 030 pounds.
Moment of inertia	262.5.	Moment of inertia	76.
Distance of neutral axis from base	3.34 inches.	Distance of neutral axis from base	2 inches.

Both lintels were subjected to a center load. When 81 700 pounds had been applied to lintel "A," its deflection was approximately ¼ inches. Upon releasing the load an approximate set of ⅛ inches was found. The uncertainty of manner and point of failure made it difficult and dangerous to take accurate observations. It failed under a load of 98 450 pounds, the web yielding sideways, a triangular piece finally crushing out, as shown in photograph Plate XXXVI. By the "common theory" the tension in the extreme fibers at rupture was 20 700 and the compression 52 400 pounds. If the modulus of rupture (31 400 pounds) found in the case of the inch square specimens is used the center-breaking load of this lintel should be 150 000 pounds. While this comparison is unfair, since the modulus varies with the cross-section and span, it is plain that the lintel could have sustained a greater load if the web had been stiffened sideways. In case of a building such stiffness would have been afforded by the surrounding brickwork. The fractured section was without flaw.

Lintel "B," tested on a span of six feet, broke at 43 030 pounds. The tension at failure was therefore 20 400 pounds and the compression 61 100 pounds. If 31 400 pounds is taken as the modulus of rupture, it should have sustained 66 300 pounds. In this case failure took place by tearing of the lower flange, the web having been stiffened by triangular brackets. The fracture showed two cinder spots starting from the lower edge of the flange. This reduced the effective section by from ⅛ to ¼ of a square inch. The loss in strength, however, was probably

Digitized by Google

greater than the loss in section. Lintel "A" broke in the center, its load being applied practically at one point. Lintel "B" broke at the flaws about 6 inches from the center, its load being distributed over the web for a length of about 8 inches.

All the various specimens tested gave fine grained fractures remarkably free from flaws. The latter, therefore, except in the case of lintel "B," did not influence the results. They were readily nicked at the edges by a hammer, or by striking one upon the other.

Deductions from tests on small specimens can only be approximate because of the comparatively greater influence on small castings of defects, errors of observation and measurement, and the coarser grain and poorer workmanship found in large castings.

LOUIS DE COPPET BERG, Assoc. Am. Soc. C. E.—I trust it may not seem presumptuous for an architect to speak on this subject, for I think it will be admitted that, if we leave out of the question the consideration of car wheels, what with machinery foundations, lintels and columns, an architect uses cast-iron quite as frequently as does the engineer.

Now, I sympathize with any one attempting to ameliorate anything, but, from my experience with contractors, I think Mr. Johnson will first have to change human nature before we can indiscriminately specify his superior cast-iron.

To use his specification would necessitate careful tests or inspection at the foundry. This we cannot afford out of the meager commissions paid us by clients; and were we to call upon them to bear these extra burdens, I fear they would object to paying for tests or inspections, which they would consider in the interests of science rather than in their own.

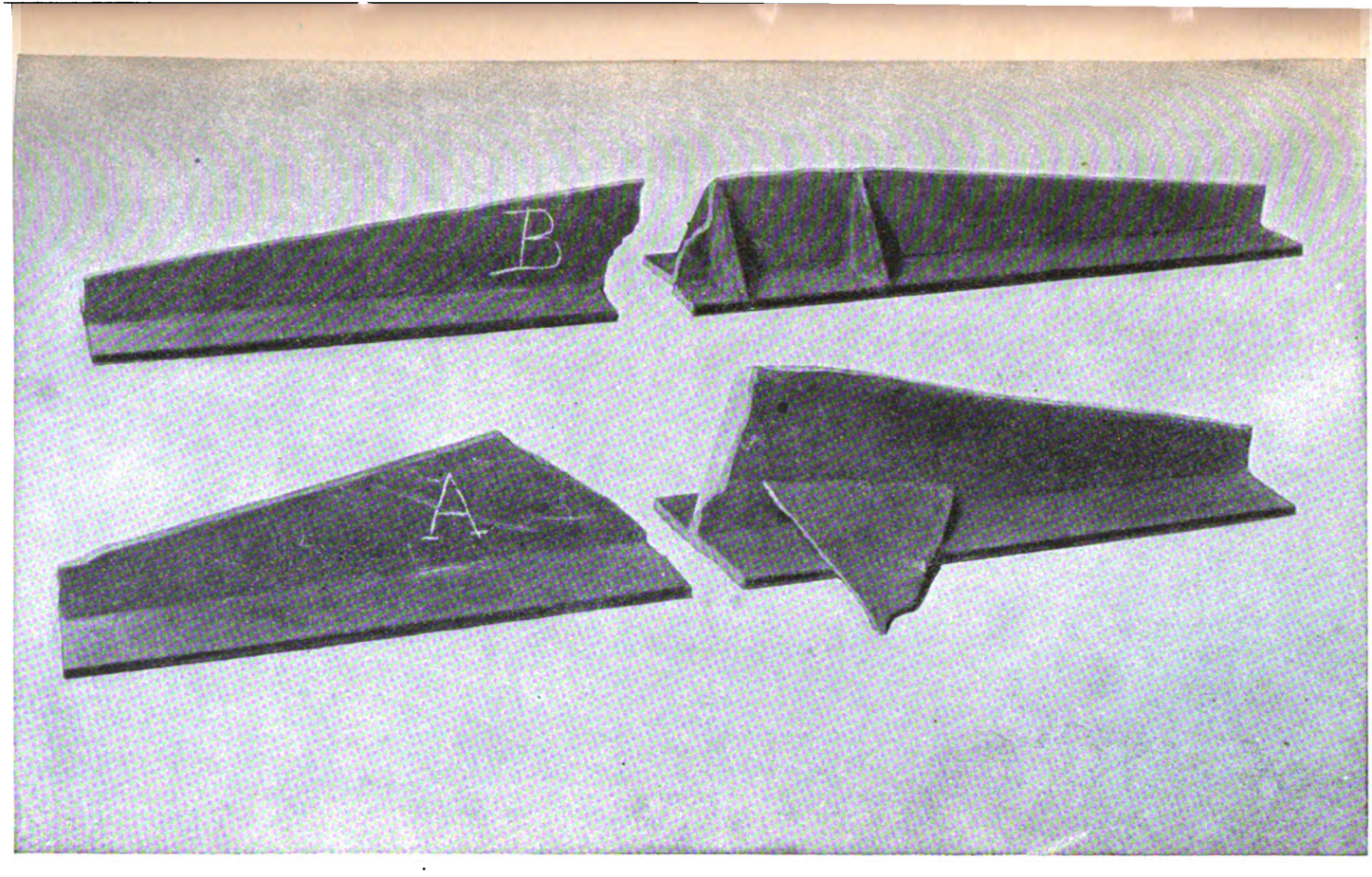
You must remember that, while a railroad company is a constant user or customer, a man who builds a house generally builds but one, and then retires from such operations a wiser and, perchance, a sadder man.

It is my rule, therefore, to call for only such quality as I am sure to get from almost any one, *i. e.*, to call for only the ordinary shop or foundry practice; and, where great reliability is desired, to use additional material.

Accordingly, in cast-iron I am satisfied with 15 000 pounds in tension and 90 000 pounds in compression (for short blocks) per square inch. Any ordinary iron will surely give this.

For beams or lintels I proportion the cross-sections, so that the center of gravity is at one-seventh of the height from the base, and then calculate the moment of resistance for the part below the neutral axis with an extreme fiber strain of 15 000 pounds, or the part above with an extreme fiber strain of 90 000 pounds. Either result is the same, the one calculation being a check on the other.

Digitized by Google



Vol. XXII. p. 128. JUST ON CAST IRON. PLATE XXXVI.

Digitized by Google

Digitized by Google

In this connection it has occurred to me that what Professor Johnson (on page 99) calls the "first error in the common theory of cross-bending," viz., the fact that the cross sections of his bars, which are plane before bending, are no longer plane after being bent in his experiments, may be accounted for by his use of rectangular sections for cross-breaking in cast-iron, instead of proportioning the section so that the neutral axis will be located at a distance from the extreme fibers approximately proportionate to their respective resistances to tension and compression. It seems to me that it would be well to make careful cross-breaking tests on such proportioned sections before condemning the common theory of cross-bending.

I do not like to use a modulus of rupture for cast-iron, particularly one so high as 36 000 pounds, on which Mr. Johnson's formula is based; for that, with a section uniform above and below the neutral axis, would be placing cast-iron on a par with our calculations of wrought-iron beams. Again, Mr. Johnson's formula makes no allowance for thickness of casting, which, as we all know, very materially affects the results. I would suggest that the specification should have these clauses added:

"All hollow castings to be bored on four sides in the same plane (approximately) at about half the length, to ascertain the uniformity of thickness," and

"All sudden changes in direction to be made by rounded corners, and not by sharp angles."

Tests other than the hammer test are of little value, unless the test metal is taken from the same melting, and is generally of the same size, and particularly of the same thickness as the casting. The resilience test may answer theoretically, but its practical value I doubt very much. We should have to educate our foundry men to a very much higher pitch, and I fear that many of our most eminent architects might also require considerable brushing up to comprehend it.

In looking up the Watertown Arsenal tests on gun iron, which, I take it, represent the most careful castings possible, I find that all our work would have to be at least equal to them if these specifications are to prevail.

Nor do I find any such discrepancies between dressed and undressed iron, as Mr. Johnson gives. The statement in the report to the Government, in 1885—that the slight gain in strength of dressed cast-iron is presumably due to the "removal of defective surface metal," is probably the true solution of the apparently greater strength of dressed cast-iron; but we must remember that if the skin does not add to the strength, it adds to a more important quality, viz., the durability of iron by protecting it from rust.

J. B. JOHNSON, M. Am. Soc. C. E.—There are two subjects treated in my paper which are both new and important. On neither of these has there been offered any discussion. They are: A rational explana-

Digitized by Google

tion of the discrepancy between the tensile and cross-breaking moduli of cast-iron, and the extreme importance of and the simple rule for finding the resilience of cast-iron.

There has never been offered, so far as I am aware, anything approaching a rational, sufficient, and consistent explanation of the high cross-breaking modulus of cast-iron, neither has the subject of the resilience of cast-iron ever received due consideration at the hands of engineers.

The former subject should certainly be of interest to scientific men, and the latter is of extreme importance to all users of cast-iron where it is subject to shocks or blows.

I believe cast-iron, which has been driven out in disgrace from nearly the whole engineering field, is destined to return to the scenes of its first conquests in high honor and with marked effects.

If engineers would learn to test cast-iron for those qualities required of it in structures, and if manufacturers would learn to obtain these qualities in a high degree, which I am sure can be done without appreciably increasing the cost of production, then the time will have arrived for returning to cast-iron for a multitude of purposes where wrought-iron or mild steel only is now used. Even in car wheels, where the service demanded is purely a question of resilience, such tests are made only on the wheels themselves and not on the metal composing them. But these tests on the wheels are impact tests, by a succession of blows, which I have shown is not, and cannot be, a true test of the character of the material or of the wheel itself.

I have not only shown the importance of the subject of resilience, but have indicated, by an exceedingly simple rule, how it can be obtained. The apparatus for making this test is also very inexpensive and every foundry should have one.

When cast-iron is made and tested with intelligence, I believe there is no good reason why the superior grades may not be used for all kinds of structural purposes in the parts which act as compression members.

The old English cast-irons, tests on which form the basis of all our tables, were very weak and brittle. Their resilience was not more than about one-tenth as much as is now obtainable anywhere in this country, or there, either, at this time. It was the very poor quality of this metal which has brought cast-iron into disrepute. Had it not been for the architects, cast-iron would to-day be almost entirely excluded from structural service. To their credit it must be said they have not deserted their best friend, because he had obtained a bad name among the engineers. An engineer would to-day lose caste and be disgraced if he proposed to use cast-iron in a structure for any purpose outside of washers and bed-plates. This will not long remain so.

Referring to the discussion by Mr. Just, I can say that a tensile strength up to 30 000 pounds per square inch may be obtained without

Digitized by Google

any sacrifice in working qualities. The addition of silica to the iron softens it and increases its resilience.

The formula given in the proposed specifications for cross-breaking is a minimum value and will correspond to a tensile strength of about 20 000 pounds. There is no fixed relation between the tensile and cross-breaking moduli, even in solid sections of the same dimensions; much depends on the amount of internal strain left in the casting, and this in turn depends on the amount of shrinkage the iron undergoes in cooling. The relative values of these moduli also depend on the amount of distortion the iron takes before rupture.

Concerning Mr. Just's tests in compression, I would say his test specimens were both too short and too long to give true compressive strength.

All tests in compression should be on specimens whose height or length is twice their least lateral dimensions.

Theory shows that when friction is neglected the planes of fracture have an angle of 45 degrees with the direction of the force in direct crushing. It may be shown, however, that, calling ϕ the angle of repose or the angle whose tangent is the co-efficient of friction, the plane of rupture should have an angle of 45 degrees $+ \frac{1}{2} \phi$ with the plane of the bed. This will usually give to this plane an angle of about 63 per cent. for stone or brick, and of 60 per cent. for cast-iron when measured from the plane of the cross-section. It follows from this that a test specimen should be about twice as high as it is wide to allow the shearing forces which cause rupture to act along their normal planes. This is also a subject which I have never seen properly stated. It is time engineers ceased to use formulas and conclusions which are true only when friction is neglected—when they know that friction is one of the principal resistances which it is their business to contend with.

If there were any authority competent to decide questions pertaining to standard tests of engineering materials, one of their first acts should be to prescribe that all compressive tests should be made on specimens having a height of twice the least lateral dimension.

Anything longer than this is a column, and anything shorter gives a deceptive show of strength always greater than the real strength.

Mr. Just's tension tests were evidently made on cast-iron bars in the rough. Although the Fairbanks machine is provided with universal joints for its grips, still the friction of these when pulling hard is sufficient to prevent a perfect alignment. It is said by bridge engineers that there is never any slipping of eye-bars on pins when the bridge is deflected by a load, and that the joint acts exactly as it would if the members were riveted together. If the bar is warped, therefore, or out of line when put into the machine, there will remain a large binding moment on it unless the grips swing freely as on knife edges.

This moment will be greatest near the grips, and this probably

Digitized by Google

explains why these test bars all broke near the grips. These bars would probably have shown a much higher strength if they had been turned down. Cast-iron specimens should always be turned at the gripped ends at least, and if it is desired to test with the skin on, then the specimen must be cast larger at the ends to allow for dressing down, and still remain larger than the body of the bar.

The transverse tests are probably correct, except that the deflections were not measured very accurately, as is shown by platting the observations.

The average resilience of these bars in cross-breaking, as computed by my new rule, is 44 inch-pounds per pound of iron. Since this value varies from ten to seventy, we see that the iron is of a high grade for architectural iron, notwithstanding its strength seems low.

A shearing test is very difficult to make on any material. It is impossible to have shear acting over a finite length without a corresponding bending moment. But if it is so difficult to make a test for shear, when the experiment is arranged expressly for this purpose, we can readily see that in actual practice a failure from simple shear probably never occurs. Shearing strength is therefore of no consequence in any of the metals. It is very important in timber, because this strength is only about one-tenth to one-twentieth of the strength in other ways. But when the strength approximates closely to the tensile and compressive strength, as it does in metals, failure never occurs from shear, and hence we need make no examination of the shearing strength.

Mr. Just's suggestion to strengthen cast-iron brackets in columns by providing interior webs is a good one, but difficult of execution. I think a better provision would be to use outside lateral webs, reaching both above and below the bracket.

Referring to the remarks by Mr. Berg, I would say that it is hardly worth while to specify anything concerning the character of the iron in architectural work if no effort is to be made to learn whether or not the specifications have been complied with. He could also be able to convince his employer, doubtless, that it would be cheaper to pay a hundred dollars for inspection to insure obtaining 20 000 pounds iron and buy only two-thirds as much of it, than to calculate on getting 15 000 pounds iron and design accordingly. In fact, no intelligent designing can be done without tests to determine the strength of the material used. Good engineering as well as good architecture means a sufficient degree of safety with the greatest degree of economy. When economy is left out of account it ceases to be professional work, and becomes merely a mechanical product.

My specifications may not apply to all parts of the country. That they are not too high for the St. Louis practice I am satisfied. It may be that we are more fortunate than other regions. But the more recent reported tests of "ordinary foundry" iron in England show so much greater

Digitized by Google

strength and resilience than the older tests that I concluded foundry-men had learned something, and that this improvement was probably general. If the New York practice is behind the rest of the world in this particular, the sooner the engineers and architects of that city begin to specify better results the sooner they are likely to get them. Of the hundreds of tests of cast-iron I have made in St. Louis, I think I have never had a test as low as 16 000 pounds, except where there was a great flaw or fault of some kind. These tests have covered the whole range of ordinary foundry practice.

Mr. Berg thinks I should not question the ordinary theory of cross-bending, provided it should be found to apply to a given kind of cross-section, such as he claims to use. Now I am at liberty to question this ordinary theory if it fails to apply to any cross-section. It does in fact fail in every cross-section. With rectangular sections it fails by 100 per cent. That is, the real strength is about twice the computed strength by the ordinary theory. For web and flange sections it much more nearly applies, but never very rigidly.

Again, he feels that he has offered a legitimate objection to my proposed formu'a by saying it "wou'd be placing cast-iron on a par with our calculations of wrought-iron beams."

Now, if the tests show that good cast-iron beams are on a par with wrought-iron ones, up to the elastic limit of the latter, which is really failure for all structural purposes, then he should not criticise me or my formula, but should deny or object to the facts themselves.

Mr. Berg's rule for proportioning cast-iron beams is a correct one in theory, but if he carries it out in practice he must needs insert several cross-braces to his slender web to prevent its buckling, as Mr. Just's experiment showed it is inclined to do. In this experiment the extreme compressive fiber was less than three times as far from the neutral axis as the extreme tension fiber, whereas Mr. Berg puts his six times as far away. His beam would therefore be immensely more likely to buckle than that of Mr. Just, which did fail in that way, and that very much below its computed strength.

There is no question but this matter of lateral stability of deep central webs in lintel plates has been commonly overlooked. The brickwork would probably not hold it up to its work to the breaking point.

My discussions and rules for resilience have no application to architectural work. Here static strength alone needs to be considered.

Mr. Berg thinks my cross-breaking requirement is too high, and "equal to the Watertown Arsenal tests for gun metal." In this he is mistaken. These tests show a cross-breaking modulus as high as 75 000 pounds, and a tensile strength of over 30 000 pounds. I have said that a cross-breaking modulus of 36 000 and a tensile strength of 20 000 should be required. That the Watertown tests do not show a

Digitized by Google

greater increase of strength when the skin is removed proves simply that the unequal shrinkage and consequent internal strain was small in these test specimens, which we know must be the case in metal cast in heavy masses and subjected to great shocks.

Mr. Cooper's specification for cast-iron in cross-breaking is nearly 12 per cent. higher than mine, but from my rather extended experience I think this is too high for ordinary good foundry iron. If extra strong iron is required, then as high a cross-breaking modulus as 45 000 may be safely prescribed.

Digitized by Google

LOUISVILLE CEMENT.

The undersigned is General Agent for the following Standard Brands of Louisville Cement:

FALLS MILLS (J. Hulme Brand),
BLACK DIAMOND MILLS (River),
SPEED MILLS,
FALLS CITY MILLS,
QUEEN CITY MILLS,
ACORN MILLS,
BLACK DIAMOND MILLS (Railroad),
FERN LEAF MILLS,
EAGLE MILLS.

This Cement has been in general use throughout the West and South since 1830, most of the public works having been constructed with it. Orders for shipment to any part of the country, by rail or water, will receive prompt and careful attention.

Sales for 1888, 1,202,118 Barrels.
Sales for 1889, 1,338,464 Barrels.

WESTERN CEMENT ASSOCIATION,

247 W. Main St., Louisville, Ky.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

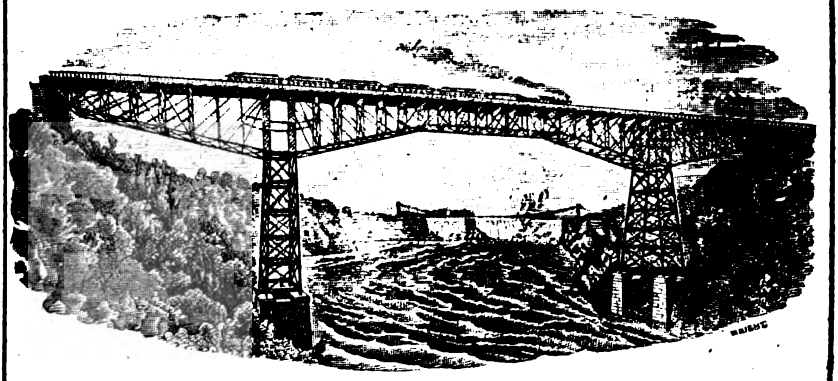
SELECT ADVERTISEMENTS WILL BE RECEIVED AT THE FOLLOWING RATES:

	TWELVE MONTHS.	SIX MONTHS.	THREE MONTHS.	ONE MONTH.
One Page.....	$100.00	$55.00	$30.00	$12.00
One-half Page.....	55.00	30.00	18.00	7.00
One-quarter Page	30.00	18.00	10.00	5.00
One-twelfth "Card".	12.00	.....	.....	....

Address the Secretary of the Society, 127 East 23d Street, New York.

Digitized by Google

UNION BRIDGE CO.



C. MACDONALD, GEO. S. FIELD,

C. S. MAURICE, EDMUND HAYES,

CIVIL ENGINEERS,

And Constructors of Iron and Steel Bridges, Viaducts, Roofs, Elevated Railroads, Marine Piers, etc.

GENERAL OFFICE,

No. 1 BROADWAY, NEW YORK.

Works: ATHENS, Pa.

Digitized by Google

MAY 12 18. VII 57c

MARCH.

1890.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Library.

CONTENTS.

TRANSACTIONS. VOL. XXII.

No.		Page
433.	The Improvement of Railway and Street Railway Track. E. E. Russell Teatman	135

ILLUSTRATIONS.

XXXVII.	Rail Sections, Joints and Bolts	208
XXXVIII.	Rail Sections, Tie Plates, Joints and Braces	208
XXXIX.	Tie Plates, Fastenings, Joints, Bolts, Sections of Road Beds, Bridge Floors.	208
XL.	Effect of Joints on Bolts	208
XLI.	Street Railway Rails and Sub-structure	208

The Transactions of the Society will be sent to any subscriber at the rate of $10 per year; and to clubs of ten or more, when ordered through the Secretary of an Engineering or Technical Society or Club, who will be responsible for the payment, at 25 per cent. discount.

PUBLISHED AT THE HOUSE OF THE SOCIETY, 127 EAST TWENTY-THIRD STREET, NEW YORK, ON THE SECOND WEDNESDAY OF THE MONTH.

Entered according to Act of Congress, in the year 1890, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Digitized by Google

American Society of Civil Engineers.

OFFICERS FOR 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN, WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Standing Committees.

On FINANCE :—Charles B. Brush, Mendes Cohen, Robert Van Buren.
On LIBRARY :—Alphonse Fteley, Theodore Voorhees, William Ludlow.

Special Committees.

THE PRESIDENT OF THE SOCIETY IS HONORARY MEMBER OF ALL COMMITTEES.

ON UNIFORM STANDARD TIME :—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston, Fred. Brooks.

ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY :—F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, A. V. Abbott.

ON UNIFORM METHODS OF TESTING MATERIALS USED IN METALLIC STRUCTURES, AND ON REQUIREMENTS FOR THESE MATERIALS TO FURTHER IMPROVE THE GRADE OF SUCH STRUCTURES:—G. Bouscaren, William H. Burr, J. G. Dagron, R. W. Hunt, H. G. Morse, P. Roberts, Jr., H. B. Seaman.

MEETINGS.

Regular Meetings, on the first and third Wednesdays of the month, at 20 o'clock, except during July and August.

The rooms of the Society are open from 9 to 17 o'clock each business day, except Saturday, when they are closed at 15 o'clock.

The Library and Conversation Room will also, for the present, be open every *Wednesday Evening*, from 19½ to 22 o'clock. Members are invited to avail themselves of the opportunities afforded on these evenings, both for consultation of books and periodicals, and for conversation.

HOUSE OF THE SOCIETY—127 EAST TWENTY-THIRD STREET, NEW YORK.

Digitized by Google

MAY 12 189

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

433.

Vol. XXII.—March, 1890.

THE IMPROVEMENT OF RAILWAY AND STREET RAILWAY TRACK.

By E. E. RUSSELL TRATMAN.

At the Convention of the Society in 1888, I had the pleasure of presenting a paper on "English Railway Track" (*Transactions*, June, 1888), in which, after drawing comparisons between the American and the English systems of railway track, I expressed my opinion as in favor of the American system, and gave the reasons for this opinion. In the discussion of that paper I modified the statement by saying that it was the American type of track as it should be, and not the actual American track as it too often exists in practice, to which I referred in advancing the opinion above mentioned, which opinion has since been considerably strengthened.

Some investigations as to the difference between the track as it is and the track as it should be, showed that there was a steady, if slow, tide of improvement, and it therefore occurred to me that some notes on the subject set forth in the title of this paper might be interesting to the Society as showing the general tendency towards a better standard of efficiency in track and track equipment, and showing also the advan-

Digitized by Google

tages following upon this tendency. It is also hoped that by describing and suggesting different improvements, this paper may be of some service in arousing interest in this matter and so helping towards further improvements in this important direction, improvements which would benefit the railway companies and the public, and tend to advance the already high status of American railway engineering.

In preparing the paper I had in view the purpose of opening a discussion on the subject, as experiments with new devices and improved methods in various matters relating to track are all the time being made by railway engineers, superintendents, road-masters and others in different parts of the country, but the particulars are often not known to others interested who may be working, or trying to work, in the same line.

In this connection I would call attention to the advantages to be derived from the meetings, discussions and publications of the several associations of practical railway men, as actual service only can test any improvements in railway track, and it is the men who have experience with these improvements who can give the most valuable information respecting the results. Persons taking an active interest in these matters should study the technical papers and the publications of the engineering and railway associations. It is also a good plan to procure manufacturers' circulars and catalogues, as they often contain much useful and interesting information, especially in regard to new devices.

In a brief paper on "The Improvement of Railway Track," read before the American Society of Railway Superintendents, in April, 1889, I took the opportunity of suggesting that one of its committees should be directed to consider and report upon the following matters:

First.—The merits of different forms of rail sections, and the adoption of standard sections from 50 to 110 pounds per yard, with a recommendation for the introduction of heavy rails.

Second.—The merits of different methods of securing rails to ties, and the recommendation of some method as an improvement on the now universal method of spiking, especially for heavy rails.

Third.—The merits of different rail-joint fastenings, and the recommendation of a standard joint fastening, with due consideration of the recommended standard tie fastening.

Fourth.—The question of the advantages of steel ties for main lines with heavy traffic, and for prairie lines, with the desirability of making any recommendations on this point. Also the consideration of the use of metal tie-plates on wooden ties.

Fifth.—The equipment of railways with such safety appliances as the block system, complete systems of semaphore signals, interlocking apparatus, frogless and safety switches, re-railing guards, the use of various electric and automatic signals, etc.

Digitized by Google

General Remarks.

I have already said that many improvements are being made, but on the whole the progress of the improvement of the track is far behind that of the motive power and rolling stock equipment. The size, power and weight of the locomotives, and the size, weight and capacity of the cars have been increased enormously; the fitting and furnishing of passenger rolling stock has been brought to a high degree of elegance and convenience, while at the same time safety appliances have been very extensively introduced, so that passengers may ride in comfort and safety, as far as the trains themselves are concerned. The improved construction of the engines and cars is an economy for the railway companies, and economy also results from the introduction of automatic couplers, continuous brakes, safety heating and lighting apparatus, and other modern improvements. The increase in the weight of the rolling stock is usually carried on with little regard to the capability of the track to safely carry these weights, as the rolling stock and track departments are usually quite independent of each other. Considering the important relations between the two, however, it certainly seems that some measures should be taken to insure a conference between the departments when any extra weight or work is to be put upon the track, so that the latter may, as far as possible, be put in condition to sustain it.

The track, however, is very generally improved only in so far as it must be; and although the improvements which are being made from year to year by different individuals mark a certain rate of progress, yet the average rate of improvement of the track has by no means kept pace with that of the rolling stock. It certainly seems inconsistent to run trains of heavy, elaborately furnished and decorated cars, hauled by heavy engines of first-class construction, over a track consisting of 56 to 70-pound rails, more or less securely spiked to the ties, and having, among other deficiencies and dangers, numerous switches, which break up the main track, but are left practically or actually unprotected. An illustration of this, which I mention because it was within my own experience, was a night accident on the Portland and Ogdensburg Railway, in Maine, in 1886. The train was a south-bound express from Montreal and consisted of baggage car, ordinary passenger cars and parlor cars. The track was fairly good, and the train was running at con-

Digitized by Google

siderable speed, when the engine struck a trailing stub switch which had been left set for the side track by the crew of a freight train which had used this side track during the afternoon. There was thus a break in the continuity of the main track, and the wheels dropped off the ends of the rails onto the ties; the engine fell over on its side and the tender smashed the cab; the baggage car went up the side of a shallow cut and into a field, and the first passenger car was stripped from its trucks and came to a stop with the front end resting on the slope of the cut. The engine driver and fireman were both killed. With a misplaced split switch, as a trailing switch, no accident would have occurred, and the only probable damage would have been the breaking of a switch rod; while even had the stub switch been protected by signals or an ordinary switch light, the engine driver might have been able to stop or reduce the speed of the train and prevent the accident. The Railway Commissioners of Maine probably had this accident in mind when, in their report for 1886, they recommended the enactment of a law requiring every road in the State that had trains running at night to place safety switches and lights at every siding which such trains pass without stopping. This recommendation was good as far as it went, but it did not go far enough; accidents of this kind are liable to occur during the day, and they are only one variety of a large class of accidents. The railway authorities in Australia take especial care to prevent accidents of this character, as will be seen further on, under the head of "Signals and Interlocking."

The above accident was one of minor importance comparatively; but how frequently does the daily press report cases of trains of all classes, express and local, passenger and freight, being turned off the main track by a misplaced switch, with more or less serious results! The switch is only one point of danger, the frog is another, and is responsible for many accidents; while slight or serious accidents are constantly being caused by broken rails, bad joints, or by the spreading or overturning of the rails. Defective joints, sometimes with one or two of the bolts loose or dropped out; and defective fastenings, with loose or worn spikes, are only too common, and, as stated in my paper, read at the Convention of 1888: "Any one who cares to make a little observation can learn some very startling facts on the matter of rail fastenings."

Hundreds of the accidents which occur every year are attributable to deficiencies of the track. The familiar claim is made that railways can-

Digitized by Google

not afford the expense of laying or relaying their lines with first-class track; but on the other hand they continue to put on heavier engines and more elegant cars, and say that the public demands extra accommodations. I am fully alive to the fact that the public is a very unpleasant factor in railway economics, being an unreasoning body which wants a maximum of convenience at a minimum of cost, and wants especially things that it can see and appreciate, and that appeal directly to its senses. With two railways between the same points, one line advertising the excellence and safety of its track, and running ordinarily comfortable cars, and the other advertising the elegance and comforts of its cars, and running its trains over light rails and an inferior track, the latter would probably be the better patronized. It is interesting to notice, however, as one of the signs of the times, that some of the principal railways make allusions in their advertisements and time-tables to the excellence and safety of the track as well as to the elegance of the rolling stock. It certainly seems that there ought to be some way, and that there must be some way waiting for the right man to find it and develop it, by which railways will be able to lay and maintain a good, safe track, keep up a favorable reputation with the public, and still be good paying business concerns. Such a way will probably be found when the Government assumes some controlling power, or when the rich railway corporations—and there are many sufficiently rich to do this—give their engineers and superintendents authority to make proper expenditures, at their discretion, in the construction, maintenance and equipment of the track, so that these responsible officials can keep the track laid with heavy rails in good condition; try different forms of fastenings and joints on a sufficiently large and practical scale as to arrive at some form that proves practically efficient; adopt safety switches and frogs, put up complete systems of signals, fence in the tracks, and in other ways improve the track and track equipment, without being called to task for the temporary increase in the track accounts. With such expenditures the road will be safer and better; the continuous drain of maintenance expenses will be reduced, and the liability of the company to damages by reason of accidents, reduced to a minimum.

It too often happens that economy is rigidly enforced in track work, even to the extent of parsimony, while at the same time money is being spent lavishly in gorgeous cars, in expensively built and fitted station

Digitized by Google

buildings, in unnecessary advertising, and in rate wars which are in the end detrimental to all the roads concerned, and which tend to increase the popular prejudice against railway companies. If any particular economy is to be exercised, let it be effected in the rolling stock or traffic department, or in the buildings, but last of all on the track. In speaking of the rolling stock, I mean, of course, in its decoration and furnishing, as it is, of course, economy to keep the engines and cars in good condition. Many of the cars are already too magnificent in their fittings; very few of the passengers demand or appreciate such ultra elegance in style, and in fact the decoration and furnishing are in many cases carried to an unnecessary and inartistic extreme. The car builders are partially responsible for this, but the main responsibility rests with the railway companies which authorize the excessive expenditures in this direction, while cutting down expenses on what are actual improvements to the company's property. The gilding, painting and carving of a palace car will not afford any protection when the car is derailed at an unprotected switch, bad joint or loose rail. Of course, the elegance of the cars is worked as an advertisement, but there is a limit to reasonable advertising of this kind; it seems that this limit has already been passed, and that cars fully as comfortable and convenient as some of those now in use, and sufficiently handsome for all reasonable purposes and to meet the demands of "aristocratic" passengers, can be obtained at prices considerably less than those now paid. Handsomely built and fitted stations are much to be desired, and tend to increase the popularity of the line, but such work should not be undertaken while the track is in any but first-class condition. I do not intend here to go further into this matter of relative economy, but hope the suggestions and arguments presented may fall on good ground and bring forth fruit abundantly.

Government Regulation of Railways.

In view of some of the modern developments in the progress of railway affairs in this country, it seems quite probable that some kind of legislative action may be taken in the future to give the Government some direct power over the railways, to provide for certain measures in the interest of the traveling public, and to insure the carrying out of these measures. Whether such action is desirable or not is a matter of opinion, but the idea is certainly not visionary, as

Digitized by Google

some of the State governments have already taken action in regard to safety appliances for trains. The Interstate Commerce Commission exercises a Government supervision over matters relating to railway rates and traffic, and has recently been obtaining statistics and making recommendations in regard to the use of continuous brakes for freight trains, automatic couplers, etc. The Government regulation referred to will probably be through a Railway Commission or Railway Department charged with the inspection and regulation of railways in regard to their construction, equipment and operation, and empowered to enforce the carrying out of the recommendations of its engineers and inspectors. The work of this Commission or Department should also include the keeping of records and statistics for publication, the investigation of accidents, collecting of statistics and information in regard to the same, and the recommending of means to reduce existing dangers. The growing tendency towards the consolidation of the railways into systems under one general management is a matter closely related to this question, as the best results as to efficiency of service, safety of operation and financial returns, would probably be obtained with the railways divided into a comparatively few systems, owned by private companies, and under the general supervision of a Government commission. If objections are raised against the practicability of such a commission, it is only necessary to look back a few years, and recall the claims made that the Interstate Commerce Commission could not continue, and that it would be practically powerless and useless. In the hands of upright and intelligent men, fitted for the duties imposed upon them (and men of this character are essential in work of such extent and importance), the Commission has proved both powerful and valuable, and is in many ways paving the way for the future Railway Commission. The latter would be organized somewhat on the plans of the State Boards of Railway Commissioners and the English Board of Trade, but with much greater power and larger scope than any existing organization. Its work would include the approval of bridge designs, the proper construction and equipment of the track; maintenance of track, bridges and buildings in good condition; train service and rolling stock equipment; methods of operation, etc. New work would be under its general supervision; and periodical inspections and reports would be made of existing lines and equipment. In regard to accidents it would be somewhat similar to the Board of Trade, in England, whose

Digitized by Google

engineers make investigations and submit reports, making recommendations and awarding the blame. The Commission would also investigate complaints as to train service, faulty construction or equipment, etc. For the first few years the work would be very heavy, but the results would probably be very beneficial.

Rails.

Since the early days of modern railways there have been very widely differing opinions as to the forms of rails. Double-headed reversible, double-headed non-reversible (or bull-headed), bridge, flange, and compound rails have been used, and rails of various sections and patterns have been designed, some by railway men, and others by men who probably knew little or nothing about railways, but believed they could design a rail to fill a "long-felt want." The double-headed and bull-headed rails not being used in this country, and not being likely to be introduced, or to come into more extensive use in other countries, need not be herein described, but some of these sections will be found in my paper on "English Railway Track" (*Transactions*, June, 1888). The use of the bridge rail has been suggested, as noted later, and one form of compound rail, which consisted of two bars each forming one side of the complete rail, is said to have given good results on the New York Central Railway some years ago; but in winter ice formed on the head of the rail, along the junction of the two pieces, and caused the driving wheels of engines to stop. The standard type of rail section is the "tee" or "flange" rail, known in England as the "Vignoles" or "flat-bottomed" rail. In this country the T rail dates back to the very early days of railways, having been designed about 1830, by Colonel Robert Stevens, engineer of the Camden and Amboy Railway; it has been developed into all sorts of variations, but still keeping its general form of section.

Coming down to recent years we find rails of various dimensions and proportions, but differing most conspicuously in the form of the head. There have been heads with top tables of long and short radius, sharp and flat top corner curves, inward flaring, outward flaring, vertical and rounded sides; the heads themselves being wide or narrow, deep or shallow. It is not within the scope or purpose of this paper to follow up the development of the form of rail heads, but the Society is to be congratulated on having, through the work of its Committee on "The

Digitized by Google

Proper Relation to one Another of the Sections of Railway Wheels and Rails" (*Transactions*, October, 1889), brought the varying opinions down to details and endorsed a form of head which is practically approved by a majority of the leading engineers and manufacturers. An actual unanimity of opinion was not to be expected, but the practical unanimity shown by the report of the committee is both pleasing and surprising. With the appointment of a new committee to prepare complete designs for sections for rails of various weights, acting in conjunction with engineers and manufacturers, an important step will have been taken toward the general adoption of standard rail sections. The question of the form of rails has been considered very fully by the committee whose report is noted above. The type of section of head generally approved, and recommended by that committee, is broad in proportion to its depth, with a top radius of 12 inches, top corners of $\frac{1}{4}$-inch radius, vertical sides and sharp bottom corners, the general form being similar to that of the 80-pound standard rail sections of the Michigan Central and New York Central railways, except that the latter has sides flaring outwards slightly from the top.

There is one point, however, on which I would like to offer a suggestion in regard to this section; engineers and rail manufacturers are now endeavoring to get a good rail steel that will stand considerable wear, and if such steel is obtained and is rolled into rails with shallow heads, the heads would be unduly weak when partly worn, and would, therefore, detract from the general life and value of the rail. Much has been said in regard to the great extent of wear borne by the iron rails of former years. Mr. Don J. Whittemore, M. Am. Soc. C. E., in his paper on "Cylindrical Wheels and Flat-topped Rails" (*Transactions*, September, 1889), refers to the iron rails of twenty-five or thirty years ago having worn down $\frac{1}{2}$-inch, and mentions a wear of $\frac{3}{8}$-inch on steel rails when they were first introduced and lighter loads prevailed. If, then, a material can be procured and rolled which will give as great a capacity for wear under the more severe conditions of modern traffic, the depth of the rail head will be one of the most important points to be considered in designing the section of the rail. But, at the same time, the rail will have to be considered as a whole, and the proportions of its material so distributed as to give the best results on rolling. To reconcile all these various points will be no easy matter, but it may be confidently expected that the Society's committee on rail sections will be successful in carrying out the delicate and important work entrusted to it. The grades of steel used,

Digitized by Google

methods of manufacture and tests of finished rails also require consideration. For further particulars in regard to rails and the traffic they carry, see the paper by Mr. Joseph T. Dodge, M. Am. Soc. C. E., on "Destruction of Rails by Excessive Weights" (*Transactions*, April, 1889).

As to weight, there are in actual service rails weighing 80 to 90 pounds per yard, and sections of the "Sayre" rail have been prepared by its designer, Mr. Robert Sayre, for weights of 84, 92, 101 and 110 pounds per yard, the general proportions being similar to those of the 76-pound rails in use on the Lehigh Valley Railway. Besides the 80-pound rails already mentioned, the Pennsylvania Railway has rails weighing 85 pounds per yard, and the Philadelphia and Reading has some weighing 90 pounds per yard, while the Chignecto Ship Railway, in New Brunswick, crossing the isthmus at the head of the Bay of Fundy, will be laid with rails weighing 110 pounds per yard. These, however, are exceptional cases. Many roads with heavy traffic use much lighter rails; the New York, New Haven and Hartford Railway has recently adopted a 73-pound rail; Boston and Albany Railway, 72 pounds; the Lake Shore and Michigan Southern Railway has a 71-pound rail for about fifty miles west from Buffalo, N. Y., all the other parts of its main line being laid with 65-pound rails; the heaviest rail on the Central Railway of New Jersey weighs 76 pounds per yard. In increasing the weight of rails, the mass of material is not the only point to be considered; the proper and symmetrical arrangement to give correct proportions throughout and to obtain the best effects in rolling, is a matter of prime importance.

For lines with heavy traffic the minimum weight of rail should be 80 pounds per yard. It may be objected that this is going ahead too rapidly; but if the amount of the traffic, the weight of the engines and trains and the pounding of the wheels are considered, it will probably be admitted that 80 pounds is not too heavy, especially as the use of heavy rails of proper design and good quality not only gives a better track and joints, but also reduces the maintenance expenses. The New York Central and Hudson River Railway has eight-wheel engines with a load of 9 tons on each driving wheel, and engines with 10 tons on each driving wheel are expected to be brought out early in 1890. For what may be termed secondary trunk lines, 75-pound rails may be admissible until such time as the railway companies realize the advantages of a heavy track, or until the use of such track is in some way

Digitized by Google

enforced, possibly by a Government commission as suggested in a former paragraph For lines with local or ordinary light traffic, lighter rails may still be used. In looking through the reports of State railway commissioners, however, I find Southern roads with 50 to 60-pound rails over which Pullman cars are run. As to heavier rails, it is not necessary to hint at a limit; but it would be very interesting to see a modified form of Mr. C. P. Sandberg's 100-pound Goliath rail tried here, with its metal tie plates and screw fastenings. A case in which it seems as though the use of heavy rails may have been carried to an unnecessary extreme, is the introduction of some of these 100-pound Sandberg rails into Victoria, Australia. It may be that the present or prospective traffic is sufficient to warrant the use of such a rail; but from what we understand of the traffic an 80 or 90-pound rail would appear to be amply sufficient. Heavy rails have been used in England for many years. In "A Rudimentary Treatise on Railways," by R. M. Stephenson, published in London in 1850, the 84-pound rail of the London and North Western Railway is described and illustrated; a 90-pound rail is now in use on some parts of that line.

It has been stated that rails 60 feet long are being tried as an experiment on the Pennsylvania Railway. Rails are frequently rolled to this length at some works, but they are usually cut to the regular 30 feet lengths. The handling and laying of heavy rails of such length would require special tracklaying or section gangs. In Europe, however, rails weighing about 75 pounds per yard are used in lengths of 39.36 feet and 60 feet rails are in use on the London and North Western Railway.

Mr. D. J. Whittemore, in closing his paper on "Cylindrical Wheels and Flat-Topped Rails," says: "If a change is necessary, why not have our form of rail a type of that noblest of all creations, a level-headed man with two legs and feet to correspond? Why not come back to the design of the American Strickland of 1834, redesigned by Brunel in 1835, and still in use on the Great Western Railway of England?" In this connection it may be noted that in 1885 Mr. C. C. Wrenshall (then of the Northern Pacific Railway) suggested an 85-pound bridge rail of the type referred to, with a 50-pound combination splice and joint plate. This rail was to be $4\frac{1}{2}$ inches high, with a head $2\frac{3}{4}\frac{3}{2}$ inches wide and about $1\frac{7}{16}$ inches deep at the middle; top radius, $6\frac{7}{8}$ inches; radius of top corners, $\frac{5}{16}$ inch. The side webs were vertical, $\frac{1}{2}$ inch thick, and the width over the base was

Digitized by Google

$6\frac{7}{8}$ inches. The tracks on the great Forth Bridge in Scotland are laid with rails of bridge section, weighing 145 pounds per yard, resting on longitudinal timbers. A rail has been invented by Mr. Patterson, of Titusville, Pa., in which the web is indented while hot, the object being to prevent creeping. It seems probable that the flange rail will continue to be the standard form of rail in this and other countries, but if any change should be made, I think the bridge rail, as suggested by Mr. Whittemore and Mr. Wrenshall, would meet with very general approval.

One interesting feature in connection with rail specifications and manufacture, and the relation between engineers and manufacturers, is the "time guarantee" system of contract in general use in Europe. Under this system a clause in the specifications provides that the manufacturer shall supply new rails to replace any which may be damaged or worn out by any cause except accidents, during a specified term of years. This term varies according to the opinion of the engineer, the weight of rail, etc.; it is twelve years for the 82-pound bull-headed rail of the Great Northern Railway (England); eight years for the 75 pound flange rail of the Dublin, Wicklow and Wexford Railway (Ireland), and six years for the 50 and 61-pound flange rails of the South Australian Government Railways. This plan certainly seems to possess some advantages; it tends to make the manufacturer particularly careful as to the quality of the rails delivered, and although it would probably lead to some increase in price, it might very likely be that the extra amount would be more than balanced by the better quality of the rails due to more careful manufacture. The varying conditions of the amount and nature of the traffic on different parts of the line would make it difficult to fix a uniform term. This guarantee system is also adopted for metal ties, the term being usually from two to five years.

Mr. C. P. Sandberg has recently stated that owing to the fast rolling in modern mills, large rail heads do not wear well, as the physical hardness is only in the surface and not in the interior of the head. He advocates a wider head, which, he claims, will increase the life of rails and tires, as the latter would not require to be turned if the rail head were about as wide as the bearing surface, while it would also add to the tractive power of the engines by offering a greater surface to the driving wheels. The crushing load on these wheels would not exceed the limit of elasticity of the steel in the wider rail head. In view of the great interest attaching to the rail question, and of the acknowledged

Digitized by Google

authority of Mr. Sandberg on this question, I give Mr. Sandberg's statement in full as an appendix, and would call attention to his remark that as good a track can be made with flange rails as with the English bull-headed rails and chairs, and at less cost, which is the point I have made in my previous paper and repeated here. (Appendix A.)

Ballast.

The various merits of different materials have been very widely discussed, and the majority of opinions appears to be in favor of broken stone as the best material for main lines with heavy traffic. A very good track can be made with gravel, but it is more dusty and requires more maintenance work to keep the track in condition. Where gravel is used on main tracks it is better to carry it out beyond the ties level with the tops of the ties, as is usually done with broken stone, instead of sloping it down from the middle of tie so as to leave the ties half or entirely uncovered at the ends; this latter method is very common with gravel ballast. Probably no better roadbed could be made than that of the standard track of the New York Central and Hudson River Railway; it consists of a 6-inch bottom course of 4 to 6-inch rough quarry spawls, covered with a course of 2-inch broken stone 12 inches deep, the ballast being brought up level with the tops of the ties.

In the iron districts a number of roads use slag ballast, which should be about as good as broken stone. A chemical action, injurious to the rail, is said to be sometimes experienced with certain kinds of slag, but inquiry of engineers and others who have had experience with slag ballast, fails to support the existence of any such action, at least to any appreciable extent. With metal ties, however, buried in the ballast, it seems probable that some such trouble might be met with, considering the number of different grades of slag and their varying chemical constituents. The pitting or corrosion would be most likely to occur with fresh slag than with slag which has been in the dump for a considerable time and exposed to the action of the weather. Steel ties have been found to corrode badly in cinder ballast, and in India they have been found liable to corrosion in certain districts where the soil is heavily charged with saltpeter. The different opinions presented in regard to the use of slag ballast and the chemical action upon rails and ties, induced me to make inquiries of some engineers using this ballast in regard to their experience, and the substance of the replies, mainly favorable, is

Digitized by Google

given as an appendix. The statement in regard to the effect upon wooden ties was unexpected, but in general the material may be said to be well adapted for ballast, and it is especially desirable on account of its economy on roads in the iron districts. (See Appendix B.)

Burnt clay is used with success on some roads; among them are the Chicago, Burlington and Quincy and the Chicago, Rock Island and Pacific railways, which obtain the material from the Iowa Clay Ballast Company, of Kansas City, at about fifty or sixty cents per cubic yard, burned and loaded on the cars. On the former road it is estimated to cost ninety-four and a quarter cents per yard in place under the track, as against $1.25 for stone ballast. (See Appendix C.)

In England a very general practice is to use a "boxing" or sprinkling of medium coarse gravel over broken stone ballast, which deadens the noise and makes a very pleasant road bed to ride over. Where gravel ballast is used, especially fine sandy gravel, a covering of ashes tends to keep the dust down; this practice is in use on some New England roads. It has been suggested that the South American practice of using soft loam and letting the grass grow over it might be tried with advantage on some Western roads in the dry regions. As long as dry weather prevails this gives a good track, but care has to be taken to provide ample drainage, and with wet weather the soil gets saturated and becomes mud, causing considerable trouble. The ordinary dirt ballast, however, as used on some of the Southern roads, requires considerable labor in maintenance, especially in wet seasons, in order to keep the track in fair condition for running.

In regard to the cross-section of the road-bed, broken stone ballast is generally brought up level with the tops of the ties, and extended well beyond the ends, giving plenty of material to resist lateral shifting; this method is also used with gravel and cinder ballast, but a more general practice with such ballast is to have it level with the top of the tie at the middle, sloping off to the middle or bottom of the end of the tie, and then rounded off to subgrade. This reduces the amount of ballast, and is intended to facilitate drainage, but it does not appear to make so good a bed, nor so efficient looking a bed, as ballast extended beyond the ends of the ties. On some roads it is so extended, and this will probably be found the better practice. The bottom drainage is the most important, as the moisture and wet soon sink through the ballast, and gravel ballast, after a short time of dry weather, even in a wet season, will soon be almost as dusty as in dry seasons.

Digitized by Google

Ties.

The principal improvements to be made in this direction are in keeping the track supplied with good, sound ties, and in increasing their durability. They are frequently left in the track much longer than is desirable after being rotted, split by spikes, deeply flange worn, or otherwise damaged, and with enlarged spike holes which give the spike a very poor hold in the wood. It is certainly economy to use ties of the best material and quality, not only to secure a better grade of track, but also to reduce the maintenance work and renewals. Suitable species of timber are not always availed of, and it is only about two years ago that the Department of Agriculture called attention to the fact that the chestnut oak, which grows in the South and is felled for its bark for tanning purposes, the timber being left to rot, is an excellent material for ties, better than some woods already used. The result of this has been that a large amount of this hitherto wasted timber has been used for ties and with satisfactory results. There may yet be other species of timber which can be used successfully for ties. The following table of oak timbers was issued by the Department of Agriculture at the time referred to. Very full statistics of the ties used on the various railways of the country will be found in the Report on Forestry, Vol. IV,

Table of Oak Timbers.

Species.	Range.	Weight per cubic foot.	Specific Gravity.	Resistance to Indentation.	Elasticity.	Transverse Strength.
White Oak, (*Quercus alba. L.*).....	East of the Rocky Mountains.....	46.35	0.7470 (4)	3388 (6)	97089 (2)	905 (4)
Chestnut or Rock Chestnut Oak (*Q. prinus. L.*)	Northeastern, and in Kentucky, Tennessee and Alabama.....	46.73	0.7499 (3)	3688 (5)	125473 (1)	1031 (2)
Basket or Cow Oak (*Q. Michauxii, Nutt.*).....	Southeastern.....	50.10	0.8039 (2)	3725 (4)	96373 (3)	1118 (1)
Burr, Mossy Cup or Over Cup Oak (*Q. macrocarpa, Michx.*). ...	Northern U. S.....	46.45	0.7453 (6)	3730 (3)	92929 (4)	982 (3)
Post or Iron Oak (*Q. obtusiloba, Michx.*).....	East of the Rocky Mountains.....	52.14	0.8367 (1)	4415 (1)	83257 (5)	872 (6)
California White Oak (*Q. Garryana, Dougl.*)	Pacific Coast.....	46.45	0.7453 (5)	3846 (2)	81109 (6)	879 (5)
Black Locust.....	Confined; but artificially almost everywhere.	54.00 (dry)	0.7333	2580	129238	1273

Digitized by Google

1884, prepared for the Department by Dr. N. H. Egleston, then Chief of the Bureau of Forestry. The present Forestry Division, of which Mr. B. E. Fernow is Chief, has also done much valuable work in connection with the railway tie question, the table here given having been prepared by Mr. Fernow.

The column of specific gravity will allow an estimate in regard to the adhesion of spikes, while the column of indentation allows an estimate as to the cutting by the rail flange.

The application of preservative processes for ties does not appear to have met with much favor thus far in this country, although very generally and successfully employed in Europe. There are several plants in operation, using different processes, but the treatment is more frequently applied to piles and timbers that will be subject to the attacks of the teredo or other destructive agencies than to ties, but the use of preserved ties is probably increasing steadily if slowly. It is said to have been definitely established as a fact that the zinc process (which is used by the Chicago Tie Preserving Company, of which Mr. Octave Chanute, M. Am. Soc. C. E., is the engineer) not only preserves but hardens the wood. The same effect is claimed for wood creosote oil. This is an especially important feature in this country, as the principal objection to the flange rail is that its comparatively narrow base cuts into the wood of the tie. Mr. F. C. Prindle, M. Am. Soc. C. E., Engineer and Superintendent of the Carolina Oil and Creosote Company, estimates the approximate increase in cost of creosoted over plain ties at about $14 and $16 per 1 000 feet when creosoted with 10 and 12 pounds respectively of wood creosote oil per cubic foot. He considers that there is a too prevalent railway policy of cheap building and minimum repairs. The Old Dominion Creosoting Works report having supplied ties treated with dead oil of coal tar, and state that a sample tie in service for eight years is still in good condition and likely to last several years more. These ties are claimed to last twice as long as the best oak or pine, and the prices are from seventy-five to eighty-five cents per tie at the works. Eppinger & Russell have also supplied ties treated with dead oil of coal tar. They state that numerous inquiries are made by railway men, but the increased first cost deters them from purchasing; ties 8 feet x 6 inches x 8 inches cost ninety cents to $1 each on the dock at the works. (For further particulars respecting ties see Appendix D.)

Digitized by Google

Steel is probably the material for the ties of the future. With such ties will come a general improvement in the track, as better fastenings will be used with them. Many objections have been urged against steel ties, but in view of the extent of their use, and the experience and steady progress of their introduction in other countries, there is little reason to doubt the practicability and economy of their use. It is to be regretted that this important matter has been so indifferently considered in this country, and that trials have not been made in order to ascertain the requirements for conditions obtaining here, but within the present year (1889) about eight hundred of the "Hartford" rolled steel ties have been made for the New York Central and Hudson River Railway; and a trial lot of the "Standard" stamped steel ties (with wooden bearing blocks for the rails) for the Chicago and Western Indiana Railway. A few others have also been tried. (See Appendix E.) The principal reasons for the little attention that has been given to this matter are probably the increased first cost (the subsequent economy not being sufficiently taken into account) and the great extent of the timber resources of the country. It has already been shown, however, pretty conclusively, that these timber resources are being destroyed to a dangerous extent both as regards the timber supply and the effects of the forests on climatic conditions. (See the report of the proceedings of the meeting of the American Forestry Association, in *Engineering News*, October 26th, 1889, and my paper on "Economy in the Consumption of Timber for Railway Purposes," read at that meeting, in *Engineering News*, November 2d, 1889.) Another reason is probably to be found in the very large number of patented inventions of metal ties, very few of which have much practical merit, whatever their theory may be; a majority of them being apparently designed by men entirely unfamiliar with the conditions to be met in service. It may be said, however, that the matter appears now to be assuming a more practical aspect. Steel, apart from its use as a means of reducing the consumption of timber, makes a superior class of track, and is therefore well adapted for use on main lines, without regard to the condition of the timber resources. The use of steel ties would considerably reduce the maintenance expenses, and would increase the safety of traffic, owing to the secure fastening of the rails. In this connection I quote the following sentence from a letter written to me by an English engineer: "The steel sleeper (tie) is essentially a sleeper for flat-bottomed (flange) rails; and until our rail-

Digitized by Google

way companies alter their rails I do not think steel sleepers will come into general use."

Some general information in regard to the use of metal track, with other matter in connection with the subject, will be found in my preliminary report on "The Use of Metal Track on Railways as a Substitute for Wooden Ties," issued by the Department of Agriculture in April, 1889. While the general introduction of steel ties is to be wished for, the necessity of improvements upon present methods of rail supports must not be lost sight of, and it is in making the best of existing conditions that the engineer frequently comes to the front.

The spacing of the ties should be carefully attended to, as trains run more smoothly on track where the ties are properly and regularly spaced than on track where they are laid irregularly. The spacing should be widest at the middle of the rail length and narrowest at the joint. The New York, Lake Erie and Western Railway requires the ties to be spaced with 10 inches between the edges of the bearing surfaces at the joints, and with an equal distance from center to center of all intermediate ties; for each 30-foot rail sixteen ties on main tracks, fourteen ties on branch roads and third tracks of main lines, and twelve ties on sidings and tracks for standing cars only; on main lines and branch roads where the long angle plate is used, or the joint supported, fifteen and thirteen ties respectively are to be used. The Louisville and Nashville Railway rules are as follows: Where the suspended joint with angle splice is used, the proper spacing of ties is 10 inches in clear between the edges of the two joint ties, and not more than 14 inches between the edges of the intermediate ties; where the supported joint is used the distance in the clear from the joint tie to either of the shoulder ties is 9 inches; the rest of the ties are to be spaced not more than 15 inches apart. The Michigan Central Railway, with 30-foot rails, laid to break joints, and with long six-bolt angle-bar supported joints, had the joint and shoulder ties spaced 19 inches center to center, 23 inches from the shoulder ties to the next intermediate tie, and 24 inches center to center of intermediate ties.

Tie Plates.

The principal method used in this country for increasing the life of ties in the track is the use of metal tie plates, placed between the rail and tie. With these plates softer species of wood may be used than are available without them, as they increase the bearing surface and prevent

Digitized by Google

flange cutting, while they add to the security of the spike fastenings so that the rails are not likely to spread. Besides this efficiency and safety, they may effect an economy by enabling cheaper wood to be used, and still give a track of equal strength and safety, while by increasing the life of the ties they effect an economy in maintenance and renewals. These plates may be divided into two classes.

First.—Those which are merely thin plates for the rails to rest upon, and which are used in connection with the ordinary spike fastenings. Of this class are the Servis and Perkins plates. The Servis tie plate is a shallow channel, made of iron or steel, about 8 x 3⅜ inches on the top, $\frac{3}{16}$-inch thick, and having flanges $\frac{5}{8}$-inch deep, which are driven into the wood in the direction of the grain. For heavy work the plate has also a rib along the middle, on the underside; this is to stiffen the plate, and to prevent it from bending under the weight of the traffic. The spikes may be driven through or outside of the plate, but the former is the better plan, as the plate, by tieing them together, makes both of them resist any lateral pressure. It is already in extensive use and has proved a practical success. The Perkins plate is similar to the above, 5 x 6½ inches on top, but rests on a thin wooden block placed on the tie, the spikes passing through the plate and block into the tie. The advantage claimed is that it avoids the cutting of the tie by the flanges of the channel tie plates. Its disadvantages are that it shortens the hold of the spike in the tie, increases the number of parts and the amount of wood used, reduces the stability or homogeneity of the track, and practically prevents shimming. The cutting of the tie by the flanges of the plates is not likely to be detrimental, as it is too slight to cause splitting, and does not offer much chance for water to lodge and cause rot.

Second.—The larger plates, which are used in connection with improved fastenings. Of this class are the Thomson (Pennsylvania Railway), the Cox (McConway & Torley), the Wrenshall, the Post and Sandberg plates, and the Stuart elastic plate. The plate designed by Mr. M. W. Thomson, of the Pennsylvania Railway, is 5 x 9 inches, $\frac{3}{8}$-inch thick under the rail, with a channel for the flange, and a rib along the upper edges of the two sides; it is fastened to the tie by screws which also hold the rail flange. The McConway & Torley plate is of malleable cast-iron; on one side is a lug which holds the rail flange, and on the other is a clamp bearing on the rail flange and a high

Digitized by Google

lug on the plate, the clamp being held by a bolt lying at an angle of 45 degrees with the vertical, with the nut on the side next to the rail. The plate is spiked to the tie. This plate is in service experimentally. Mr. C. P. Sandberg has tie plates of different forms; one form, for screw fastenings, is very similar to that of Mr. Thomson; another form is a flat plate with clips stamped up, the rails being held either by spikes or by a horizontal wedge fastening between the rail flange and the clips. Mr. Post uses on the Netherlands State Railways a heavy plate with teeth on the underside to bite into the wood and prevent slipping; screw fastenings are used. The Stuart "elastic" plate is split at the ends and so bent that the rail rests on the unsupported diagonally opposite ends of the plate, the metal being supposed to give slightly under the weight of trains. Various devices have been invented for insuring an elasticity of the track, among which may be noted a corrugated pad of rubber intended to be placed between the rail and tie. Mr. Sandberg's views on tie plates are given in Appendix A.

Tie Fastenings.

In the method of fastening the rails to the ties great improvements can be and should be made. Numerous devices have been designed for this purpose, and some of them have been given a practical trial but on a comparatively small scale. The old-fashioned spike, which is still almost universally used, is not sufficiently secure for tracks with heavy traffic, as the vertical motion of the rail gradually draws it out of the tie, in which it is held by friction only, while the lateral thrust of the rail by the wheels of trains tends to draw the inner spike and forces the outer spike back into the wood, thus enlarging the spike hole, reducing the hold of the spike and giving a good chance for water to enter and cause rot. Many of the derailments which are continually occurring are due to this loosening and drawing of the spikes, allowing the rails to be forced outward or overturned. In answer to an inquiry as to spikes for 80 or 100-pound rails, Mr. T. A. Meysenburg, of the Tudor Iron Works, St. Louis, sent me a drawing of an enlarged spike of the ordinary form which he considers suitable for such rails; it is $6\frac{1}{2}$ inches long over all, the point being $1\frac{1}{2}$ inches long; $\frac{11}{16}$ inch square under the head and $\frac{5}{8}$ inch square in the shank; the head is $1\frac{3}{8}$ inches square and has a projection of $\frac{11}{32}$ inch. Mr. Robert H. Sayre, of the Bethlehem Iron Works, the designer of the heavy "Sayre" rail sections already referred to, thinks that

Digitized by Google

"an ordinary spike with a wood screw in every other tie would be sufficient, as the rails are in all cases as broad in the flange as they are high, and there would be no tendency to overturn except at the beginning of a curve, where fast running trains would strike the head of the rail pretty hard."

Following are a few notes upon some of the "improved" spikes which have been brought out; some of which spikes have been put in service. The Davies lock spike is a thin, flat, steel spike, having the shank at an angle of about 40 degrees with the head, so that it drives diagonally into the tie, parallel with the rail, and it is claimed that in this way the rail will be prevented from drawing or loosening it, while it presents a broad surface to resist the lateral thrust; the end has cutting edges to prevent too violent a tearing of the wood, as the spike is driven across the fiber. They are about 4¾ inches long under the head, the head is 1 inch wide and 1¼ inches long; the shank is about 1¼ inches wide, tapering from ⅝ inch thick under the head to ½ inch at the end. Spikes of greater width are used at joints, and also act as rail braces on curves. Two of the spikes are driven at each rail, sloping in opposite directions. These spikes have given good results in service; among other roads using them is the New York Central and Hudson River Railway, which uses spikes 2 inches wide at the joints and some intermediate ties on a part of its line. The McLean spike is sharp edged and sharp-pointed, and the claims made for it are that it is easier to drive, cuts the fibers smoothly instead of crushing them, and so increases their hold upon the body of the spike; while the fibers, not being injured, present a better surface to resist lateral motion of the spike, and also fit so tightly around the spike that water is practically excluded. These spikes have been tried on a number of roads. The Goldie spike is sharp-pointed instead of chisel-edged, the idea being that it will separate and force back the fibers of the wood, instead of crushing them down, and will thus obtain a better hold for the spike. The Fennerty spike is of ordinary form but with grooves down the front and back faces, separating the fibers and giving a greater bearing area; these grooves may be either vertical or slightly diagonal. Some of these grooved spikes are of round section. Another form of Fennerty spike is curved slightly in side elevation, the idea being that it will hug the flange when finally driven by the last few blows, and it will offer great resistance to pulling. Tests, however, have shown that such a form of

Digitized by Google

spike is much inferior to the straight spike. The Hicks spike is a form of bayonet spike; it is of triangular section, the sides being concave and giving three sharp cutting edges; the head is of tee shape, with one side thicker than the other, so that when driven one side bears on the rail flange and the other side on the tie, acting as a brace to resist the lateral thrust of the rail. Ordinary spikes are used with the Philadelphia and Reading 90-pound rail. Numerous forms of spikes have been suggested, but an extensive series of tests made some years ago at the St. Louis bridge, have shown that it is not easy to increase the efficiency of an ordinary well-made spike, and that ragging and curving even reduce the efficiency below that of the ordinary spike.

The most effective way in which to prevent the drawing of the fastenings is to give them a positive grip or bearing, instead of relying upon side friction. This can be obtained by the use of bolts or screws. In England, the Board of Trade requires flange rail tracks to have the rails fastened at the joints and the middle of the tie by fang-bolts; spikes only not being considered safe. With the English fang-bolt fastening, the bolt is usually placed with the head bearing on the rail washer, in which case it is screwed down into a fang-nut on the underside of the tie, the nut having sharp fangs or projections which bite into the wood and so prevent turning, the bolt being turned while the nut is stationary. A better plan would seem to be to have the head of the bolt underneath, and prevented from turning, the nut being screwed down on the nail washer or clasp. Such a method I have illustrated on one of the accompanying plates, the fang washer having ribs or stops to prevent the turning of the head of the bolt; but I would further suggest as a simpler plan, also illustrated, that the fang should be on the underside of the bolt head itself, thus dispensing with washers. I have made inquiry of manufacturers and find that such bolts can be made. Mr. Sandberg uses a broad threaded screw with a wide hexagonal head which bears directly on the rail flange and the rib of the tie-plate. On the Northern Railway of France, an 85-pound flange rail is fastened by screws of a little different form to Mr. Sandberg's; but, like his, bearing directly on the rail flange, the wide part of the head is circular, with a square projection in the middle to fit the track wrench. Mr. Post uses on the Netherlands State Railway a screw similar to the above, but iron clips or washers are used on one side of the rail flange. An objection to the use of bolts is the trouble of boring the holes and putting the pieces in posi-

Digitized by Google

tion. Holes are also bored for some of the screw fastenings used, especially with hard wood ties, and this method is being used on some new railway lines being built in Venezuela by American engineers for a French company. The Bush interlocking bolts have been tried in this country, and with satisfactory results; they are in use on some parts of the New York Central and the Pennsylvania railways. Two bolts are used for each fastening, one on each side of the rail. The upper part of the bolt is at an obtuse angle with the lower part; one bolt of each pair has a projection which engages with a recess in the other bolt. The bolts are slipped into holes bored diagonally into the tie, crossing under the rail. When turned into position the bolts lock automatically. A washer bears on the rail flange and the tie, having teeth which bite into the wood and prevent any outward shifting of the rail. The nuts are screwed down on these washers and draw the interlocking arrangement tight. At joints with angle bars, the flange of the angle bar is cut away to allow the washer to bear on the rail.

All things considered, a screw fastening which does require the boring of holes in the tie seems to present the most advantages. A fastening of this kind which promises good results is that designed or adapted by Mr. M. W. Thomson, Engineer of Maintenance of Way, Pennsylvania Railroad. It is similar in form to the Sandberg screw fastening, and has a similar tie plate; but it has the advantage of using clips or washers to hold the rails, while the pointed screw can be easily started into the wood, thus expediting the track work. There are three $\frac{7}{8}$-inch screws, about 5 inches long, to each fastening. For the present it is intended to use only four of these fastenings to a rail length, which will enable the efficiency to be as well tested as with a larger number, and will reduce the cost of the experiment. The tie plates and washers can be rolled in bars and sheared to length.

Another form of fastening which is in extensive use in other countries is the horizontal taper key or wedge, but it can only be used in connection with metal ties or tie plates, the key being driven between the rail flange and a lug on the tie or plate, another lug holding the opposite side of the flange. It is very simple and efficient and has given very good results; being easy to drive and not liable to work loose, though as a precaution the smaller end of the key is split and can be opened slightly with a chisel when driven to place. In the Fennerty joint plate, an American invention, the fastening is a flat taper key, with a stud

Digitized by Google

on the small end on which a nut can be screwed to prevent it from working loose.

In order to relieve the strain on the fastenings at curves, especially where ordinary spikes are used, the rails should be connected by tie rods or gauge rods, which, by connecting the rails, would probably be more efficient than the ordinary outside rail braces. The rods may be similar to those used at switches, holding the rail by the flange, or may have angle pieces at the ends to fit the outer side of the rail flange and web; they may also be screwed through the web and secured by nuts, which method is adopted with some systems of metal longitudinals used in Europe; this, however, might be inconvenient. Since making this suggestion I have heard and read that such a plan is actually in use abroad. A translated article on "The Narrow Gauge Railways of the Lake of Lugano," in the abstracts of foreign papers in Minutes of Proceedings Inst. C. E., Vol. LXXXIII, by E. Olivieri, taken from "Il Politecno," an Italian technical paper, mentions such a practice as in use on those lines. The gauge is 2 feet 9½ inches; the track is laid with steel flange rails 4½ inches high, 29 feet 6 inches long, 45½ pounds per yard, on eleven ties to a rail length, the ties being 5 feet 3 inches long, 5 x 6½ inches section. "A good deal of difficulty has been found in keeping the rails to gauge, and the sleepers [ties] are now (1885) supplemented with wrought iron tie rods at the joints on straight lines, three to a rail on curves of more than 230 feet radius, and five to a rail on sharper curves."

Rail Joints.

As far as inventions and devices are concerned, the rail joint is to the track what the car coupler is to the rolling stock, and it would be an endless task to attempt to describe the different forms and arrangements that have been suggested and patented. With comparatively few exceptions, the usual form of joint adopted in this country for main tracks under heavy traffic is the double angle bar joint, from 24 to 48 inches long, with four or six bolts, and sometimes with an extra tie under the joint. The Savannah, Florida and Western Railway has for its 70-pound rails angle bars 48 inches long, with six bolts. The Lake Shore and Michigan Southern Railway has a joint of the same type and dimensions; and also uses a short angle bar supported joint, the joint resting on one of the ordinary ties. The Michigan Central Railway uses mild steel splice bars 44 inches long; the bolts have the Harvey "grip-thread,"

Digitized by Google

and are spaced 6, 9 and 9 inches; experience with these joints under heavy traffic has shown good results. The New York Central and Hudson River Railway uses with its standard track with 80-pound rails a three-tie joint, with angle bars 40 inches long and six bolts; the rail ends are fastened to the tie by a pair of Bush interlocking bolts or Davies lock spikes (already described under "Rail Fastenings"). With the lighter rails, a suspended joint, with short angle bars and four bolts, is used. The Chicago and Alton Railway has also a three-tie joint, with angle bars 40 inches long and four bolts; the inner bar is 1¼ inches thick. The New York, West Shore and Buffalo Railway has a three-tie joint, with angle bars 36 inches long, weighing 45 pounds per pair. They have given excellent service and were in good condition after six years' wear under heavy traffic, with passenger engines having 32 tons on four driving wheels, and freight engines having 44 tons on eight wheels. The Philadelphia and Reading Railway uses for its 90-pound rails a suspended joint, with angle bars 30 inches long, and six bolts, the outer bolts being over the middle of the joint ties, which are spaced 22 inches apart, center to center. The New York, New Haven and Hartford Railway has for its new 73-pound rail a joint of the same type, with angle bars 24 inches long, secured by four bolts. The joint ties are 8 inches wide and spaced 8 inches apart in the clear. The joint appears to hold up well under the heavy engines, but Mr. F. S. Curtis, M. Am. Soc. C. E., the Chief Engineer, is in favor of a heavier angle bar; as to the necessity of a bar of much greater length, he thinks that is still a question. The track of the cable railway over the Brooklyn Bridge is laid with three-tie joints, long angle bars and six bolts.

Compare these joints with those of European lines, on which short, but sometimes deep, plates, with four bolts, are generally used. This is due partly to the average greater weight of the rails and partly to the more secure fastenings. The joints of the Sandberg 100-pound rails have angle bars with only four bolts, but the rail ends are held down to each joint tie by two screws which pass through the flange of the angle bars. On the Northern Railway of France, angle bars with narrow flanges are used; the bars are 26 inches long, with four bolts, and the joint ties are 28 inches apart, center to center. The rail is secured to each joint tie by two broad-headed screw spikes. A secure fastening of the rail ends of the ties aids materially in making a good joint.

The Thomson joint is to be tried on the Pennsylvania Railway in

Digitized by Google

connection with Mr. Thomson's tie-plate and fastening already described. Each side of the joint is a heavy angle bar 24 inches long, projecting under and supporting the flange, but taking no bearing under the head; only two bolts are to be used. It would seem, however, as though bolts under the rail would add to its efficiency. It is safe to predict that the very use of the improved fastenings will save much trouble with the joints. The joint has also been tried with 72-pound rails on the Chicago and Northwestern Railway.

The Fisher joint has been in use for several years, and is well known to railway men. It consists of a slightly cambered "bridge," the ends of which rest on the joint ties; the upper surface has a channel for the rail flange, and the fastening consists of a single holding down U-bolt; the corners of the rail flanges are cut off to make room for the bolt, and washers are fastened down on the flanges by nuts; between the horizontal leg of the bolt and the under side of the "bridge" is a spring plate to insure tightness of the fastenings. This joint dispenses with splice bar and ordinary joint bolts, and gives the rail a continuous bearing. The Otis joint is a modification of the Fisher joint, designed to be used with ordinary angle bars. The bridge is dispensed with, and the ends of the U-bolt pass up through the flanges of the bars; a small flat plate is placed under the rail ends between the bolt and the rail flanges.

The Cloud joint is a double angle bar joint; the bars having a vertical web projecting below the rail, with bolts under the rail connecting the webs. It would seem, however, as though a bed-plate would add to the efficiency. Deep splice bars, embracing the rounded lower head of bull-headed rails, or projecting vertically below flange rails, have been used in Europe and other countries, but I think not with bolts under the rail. The Long "truss" joint consists of a bed plate resting on the joint ties, with a transverse steel plate of arch section underneath; the sides of the plate rest on two U-bolts, with long horizontal legs, parallel with the rails and depending from the side of the bed-plate between the joint ties, which are about 22 inches center to center. A short piece of angle plate is bolted to the bed plate at each side of the rail, and a single bolt put through at the joint. A later form of this joint has wide angle bars 15 inches long, with two joint bolts; on each side of the rail is a U-bolt, parallel with the rail, the ends of which are 10⅛ inches center to center, while the middle part is bent to a radius of 6¾ inches. The ends pass up through the bed plate and

Digitized by Google

angle bar and are held by bolts; a saddle piece 5⅜ inches long rests on these bolts transversely under the rail. This joint is reported to be in service and to have given satisfactory results. The Samson bar is a plain or angle splice bar thickened in the middle to prevent breaking at the point where splice bars usually fail. Similar bars, with three degrees of thickness, have been used in Europe.

The Cox rail joint, designed by Mr. Cox, of the Pennsylvania Railway, consists of a transverse clamp used in connection with an angle bar joint. The clamp is in two pieces, each with a hooked end holding the flange of the angle bar, while the lower parts meet under the rail and support the rail flange. A single bolt passes through the two pieces under the rail. This joint has been in service. The Van Wie joint consists of a bed plate and chair, with a horizontal round bar parallel with the rail resting between a clamp plate resting on the chair and the splice plate; on this round bar are three cam-shaped projections, and by turning the bar with a wrench a strong hold is obtained on the splice plate. The Fennerty joint consists of a bed plate with a lug which holds one side of the rail flange; on the other side is placed a short angle plate bearing against the web of the rail and upon the rail flange and bed plate; it is held in place by a flat horizontal wedge, driven between its outer edge and a lug on the plate; a stud and nut on the wedge prevent it from working loose. The proposed Patterson joint is another adaptation of a key or wedge fastening for joints. The Morgan joint consists of a base plate or "bridge" made of a piece of old rail laid with the head down and having the ends swagged flat to rest on the joint ties. Two ⊃ clamps are used, the legs passing through the web of the track and bottom rail; a single flat splice plate is used on one side of the track rails. The spliced-rail joint has the rail ends halved together; the inner side of one rail end has tongues which engage with grooves in the side of the other rail end, so as to prevent independent vertical play. The rails may be bolted through the webs, or may be held by two clamps which embrace the flanges, and have a thin taper key which is driven between the rail flange and inner side of the clamps. Mr. C. C. Wrenshall, formerly of the Northern Pacific, designed a joint to be used in connection with his proposed bridge rail, which consisted of a piece of rail 12 inches long, placed between the ends of the rails and securely fastened to a bed plate; the idea is that as the wear from pounding occurs just beyond the end of the rail, in the

Digitized by Google

direction of the traffic, this piece would take the pounding and could be easily replaced.

The bevel joint, in which the ends of the rails are cut diagonally, instead of square to the length of the rail, has been tried, but apparently without much success. Whether a bevel joint with one end overlapping the other has ever been tried I do not know. A device to hold the rail ends at the same level is a five-bolt joint used in India, the middle bolt passing through an aperture made by notching the webs of the rails at the ends.

The bolts used are a very important feature in the making and maintaining of good joints. In some cases the bolt gets slack by the wearing of the head, owing to the head being too small or light, while the nut might be quite tight. In riveted work the rivets are given a certain diameter and thickness of head to secure a firm bearing, but the bearing of the head of a track bolt is often very small. In this respect the tee-headed bolts appear to present advantages over the ordinary button-headed bolts. The use of bolts with the Harvey grip thread enables nut locks to be dispensed with in many cases, thus reducing the number of parts, while giving a tight and secure fastening. Such bolts are in use on a number of roads, but where a nut lock is to be used one of the best forms is the Stark lock, with which a U spring pin slides in a groove formed by a groove in the bolt and nut; by the use of eight grooves in the nut a positive lock is obtained for every one-eighth turn; the end of the spring pin is turned up to catch in the thread of the nut. Oval bodied track bolts are coming into extensive use, and this form of bolt, with the Harvey grip thread, has been adopted by the Northern Pacific Railway as the standard track bolt, after experimenting with various forms of nut locks. Mr. C. C. Wrenshall, formerly of the Northern Pacific, says: "If you want the splices to hug the rail ends with a 'death grip' during the passage of rolling stock, then avoid all elastic substances or springs of any kind; bolt up tight with a metal strain of about 10 000 pounds per square inch, and use the Harvey grip thread, so that the nut will stay where you put it and be what it seems to be, a tight fit." The Northern Pacific bolts and nuts are shown on one of the plates. The Harvey bolt has a ratchet thread, undercut on the bearing side; the nut has also a ratchet thread, the bearing side of which is at right angles to the axis of the nut. When the nut is screwed home against the angle bar the bolt threads are forced out into the nut

Digitized by Google

threads so as to fill them completely and make a tight fit. The nut has the bolt hole enlarged at the bearing face, so as to inclose and protect the bolt threads from the rubbing or pounding action of the angle bars in service, thus insuring good threads to screw the nut upon when the joint becomes loose from wear of the angle bars. The "Ideal" nut (Reading Bolt Works) for this bolt is square at one end, with the corners rounded or chamfered off towards the other end; it is said to give a better bearing for the wrench, which will not slip.

As regards broken or even joints, it may be noted that a number of roads with the heaviest traffic and best tracks have the rails laid to break joint. As regards suspended or supported joints, the three-tie supported joint is certainly giving excellent results on tracks carrying the heaviest traffic. Where suspended joints are used, however, as they are now used quite extensively, it certainly seems reasonable to suggest that some plan of base support must be given if this part of the track is to be as good and as stable as that at the middle of the rails, especially as one of the principal objects of the joint is to keep the ends of the two rails at exactly the same level. The use of a good and efficient fastening of the rails to the ties will very much reduce the trouble with the joints, as it reduces the leverage of the rail end. With an ordinary spike fastening on an ordinary track the rail end has considerable vertical play, and the splice bars and bolts are relied on to resist the strain and hold the rail ends in position. With a secure fastening, however, the leverage is limited to the distance between the joint tie fastening and the rail end.

Those interested in the subject of joints should read a paper by Mr. Max J. Becker, Past President Am. Soc. C. E., on "Why do Rail Joints and Splice Bars Break?" read in 1885 before the Engineers' Society of Western Pennsylvania.

Frogs.

Frogs are responsible for many track accidents, and the general practice of cutting the main track rails at every insignificant side track is much to be deplored, especially on lines with fast and heavy traffic. Safety frogs have been devised, in which the side track is gradually elevated so as to carry the wheel flanges clear over the head of the inner main track rails; a movable piece, operated in connection with the movement of the switch bridges the gap over the main rails when the side track is in use. In this way the main track is unbroken at the

Digitized by Google

crossing of the turnout track. The Wharton switch includes a device of this kind. Mr. Charles Price, of the Allegheny Valley Railway, has also designed a similar arrangement, which has been tried on his road. The use of these devices, however, introduces a danger in the case of trains running at high speed, as the rapid elevation of the inner rail of the side track (outer rail of turn-out curve) on the sharp curve, is liable to cause a derailment; this might be avoided by also elevating slightly the other rail of the side track, so as to present only a short ascending grade to trains leaving the main track. Devices with movable hinged or sliding rails have also been introduced to some extent. The Parsons frog consists of a pivoted piece of rail at the intersection of the track rails; this piece being long enough to reach beyond the intersection to a point where the distance between the rail heads is sufficient to allow the passage of the wheel flanges. The rail ends and the movable rail are mounted on an iron bed plate, and the pivoted piece is moved to connect one or other of the two lines of track rails. The frog is operated by a shaft under the bed plate, which has a worm screw gearing with a portion of a nut under the rail; this shaft is connected by a wire or rod with the switch stand, so that the frog and switch are operated together. The Hoyt frog consists of two short pieces of rail, arranged in the form of a V, bolted to a steel plate which slides on a steel bed plate; lugs on the bed plate limit the travel of the slide and prevent any lateral motion by the wheels of passing trains. The sliding frog is operated in connection with the switch. A frog known as the Wuerpel and Taussig automatic frog, used on the St. Louis Bridge and Tunnel Railway, consists of a movable rail pivoted at the intersection of the track rails, with the free end pointing towards the switch and connecting with the main or side track rails according to the position of the switch, with which it is connected. A point to be considered in adopting a safety frog, is whether it is possible for a wild train, or one backing down on a misplaced switch, to pass safely over the frog. Safety frogs, or substitutes for frogs, of various kinds are in experimental use, but they should be extensively adopted, especially on important lines.

Switches.

During the past few years there has been a very marked improvement in switch apparatus, and a very extensive adoption of the improved appliances; but there is still urgent necessity for their further introduc-

Digitized by Google

tion, and there is also room for further improvement in the apparatus to extend its efficiency and usefulness. Split switches only should be used in ordinary practice, and facing switches should be avoided as far as possible. The Boston and Albany Railway uses split switches throughout, and has advanced the safety of its main track by locating them all as trailing switches in the direction of the traffic. This increases the work of freight train switching, of course, as trains have to run past the switch and then back down on the side track, but this extra work is far outbalanced by the assurance of safety in the operation of fast passenger service. On other roads investigation will often show that it is possible to materially reduce the number of facing switches. All main track switches, at least, should be protected by and interlocked with distant signals, as well as a signal at the switch; and they should also be fitted with a locking apparatus to hold the switch rails firmly in position; this apparatus also should be so connected with the signal apparatus that unless the switch rails are properly locked the signal cannot stand at track clear. It should also be made impossible, by the use of a detector bar or other means, to move the switch while the train is passing over it. Several different forms of split switches are in use, and numerous devices intended to improve upon the ordinary switch are being designed from time to time. The point of the switch rails should not be too thin, and should be well housed under the head of the stock rail. The switch rails should be connected by sufficient tie rods, and the use of some such device as that in the Lorenz switch, to enable trains to trail though a closed switch, will effect an economy in repairs. The chairs for the sliding rails should be of ample size and firmly secured to the ties and stock rails. A good method for insuring the accurate maintenance of the gauge is to lay on the tie near the point of the switch rails an iron plate long enough to carry both rail chairs, the spikes or other fastenings passing through the chair and plate into the tie. Guard rails at facing switches also provide an additional safety for trains. The ballast between the rails at switches should be kept well below the level of the top of the ties, so as to minimise the danger of stones or gravel getting between the switch and stock rails, and so interfering with the action of the switch. The ballast should also be kept clear from all moving parts, such as tie rods, safety tie rods, bell cranks, connecting rods, etc. Where possible it may be well to box in such moving parts.

As to stub switches, these should be entirely abandoned for main

Digitized by Google

tracks without delay, and even on side tracks they should be gradually replaced with split switches. This form of switch is, however, still extensively used in main tracks, though it is now generally considered to be bad practice to use it in such positions.

The Parsons switch is arranged to be operated by levers on the engine, so that the engine driver can set the switches right for his train as he goes on, and if anything prevents the action of the switch the levers are caught by a track arrangement, causing the application of the brakes and the opening of the whistle valve. This device is intended to be used in connection with an automatic block system, which is explained as follows:

The Dunn safety switch, which is a combination stub and split switch, is being experimented with on the Minneapolis, Lyndale and Minnetonka Railway. There are two sets of converging switch rails, with a wrecking block filling between them. In case of the switch being misplaced, or of a main line train coming upon the switch set for the side track, the wheels run upon the wrecking block and are guided to the track rails, a groove in the head of the rail allowing the flange to pass. Satisfactory results have been reported of the trials made with this switch.

"A long freight train coming down the main track, with an express in its rear, which wishes to make a siding, is obliged, under present circumstances, to come to a full stop, and send a brakeman ahead who has to throw the switch. Then the train starts from a dead stand still, and, in endeavoring to get on the siding, is not only liable to get stuck, but in the five minutes of the time consumed is liable to be struck in the rear by the following express. Now, by this system, as the freight train comes down the main line, it sets a signal and block one mile to the rear of the siding. Coming along with full headway, the engineer in the cab of his engine throws open his switch, enters upon the siding, and not until the last car of his train is clear on the siding does he allow the oncoming express to pass the block one mile to the rear of the switch. The freight engineer running upon the siding also closes his switch automatically, leaving the road entirely clear for the express."

Whether we shall ever come to the use of a system which throws the operation of a road and its train service into the hands of the engine drivers is a question. As, however, signals and signal operators would still be necessary to insure safety in operation, it would certainly appear better to give the operators control over the switches. The above system might be applicable in certain cases, and for side tracks distant from

Digitized by Google

a signal station, but the cost of track and engine appliances would be too great for use for such few cases, and it would probably be of little advantage to give the engine drivers control of a few of the switches on a line. Distant signals should certainly be provided for such side tracks as mentioned, used in regular freight train service, but it would probably be better to manage the switch generally in a way somewhat similar to that used in Australia, as described under the heading of "Signals and Interlocking."

Signals and Interlocking.

Railways with even an ordinary amount of traffic cannot be operated with a proper degree of safety unless equipped with signals, and the greater the traffic the more urgent is the necessity for the use of such equipment, not only to insure efficient and safe train service, but also to facilitate the prompt handling of the traffic. As the majority of signals are used in connection with switches they may fairly be mentioned here. In signals, as in switches, great improvements have been made, and their use widely extended, but there is still an enormous field for their introduction, as accidents occur continually at places which should have been protected, especially at switches. Every switch on a main line should be thus protected, as they are all about equally dangerous, whether at a crowded yard or a country side track.

In this country, where trains are run close together, as in cases where regular trains are so heavy as to be sent out in two or more sections, the engine drivers are supposed to maintain an interval of time between trains. On the New York Central and Hudson River Railway the time at which each train passes a telegraph station is indicated by displayed figures, which are illuminated at night; this method has failed, however, by a train breaking down between telegraph stations, and a following train, running according to the time displayed at the last station, running into it. The Fontaine "time interval" signal consists of a pointer and graduated scale or dial; a clock work arrangement is set going by the wheels of a train passing over a lever, and the pointer registers up to twenty minutes, when it returns to zero. An apparatus consisting of a strong glass tube with a colored fluid indicating the length of time by its height in the tube, has also been devised. It has been proved, however, over and over again, and by serious accidents, that it is not possible to insure safety in the train service by the "time interval" system, as an

Digitized by Google

accident which delays a leading train will rapidly reduce the interval without the knowledge of the men in charge of the following train. An interval of space is the only sure means of preventing accidents by collision, and this interval is maintained by the operation of the block system. The line is divided into a number of sections with signals at each end, and no train is allowed to enter upon a block section until the train ahead has passed on to the next block. This is the "absolute block" system. With the "permissive block" system, the signal man may, under certain circumstances, admit a second train upon the section, distinctly warning those in charge, however, that the section is not clear. Mr. Charles R. Johnson, President of the Johnson Railway Signal Company, states in his preface to the company's catalogue, that interlocking signals were first introduced in the United States on the New York Central and Hudson River Railway. Mr. Toucey, General Superintendent, and Mr. Buchanan, Superintendent of Motive Power, recognizing the advantage of concentrating switches as much as possible, so that they could be controlled from a central point, devised an interlocking machine; the first one was put in at the Spuyten Duyvil Junction, New York City, in 1874, and remained in service until 1888. The Pennsylvania Railway sent to England for a Saxby and Farmer machine, which was put in at the East Newark Junction, on the New York and Philadelphia division, in 1875.

As to the form of signals, the semaphore type is the one generally approved, and is almost universally adopted in all parts of the world. Various other forms of signals, however, are still in use in this country. The adoption of a uniform type of signals is most desirable, and also the adoption of a uniform code of orders conveyed by the signals, as the use of different meanings for the same position of the signal arm or lights is liable to lead to accident by confusing engine drivers who have worked on different roads. A uniform color and form of the arm is also desirable. Distant signals should have forked or swallow-tail arms, to distinguish them from the home signals. It will not be easy to obtain such a uniform system as suggested, but it can be gradually introduced by adopting certain standards to which the railways will conform in time. The signals should be placed in conspicuous places, and care should be taken to insure a clear view and a good background. They should be so arranged as to fly to "danger" in the event of the switch being tampered with or forcibly opened, and also arranged so that they cannot indicate "track clear" unless the switch rails are properly set and securely locked.

Digitized by Google

Numerous attempts have been made to make the night signals more distinct and significant than the ordinary single lantern showing a light of different color according to the position of the signal arm. In the Koyl parabolic semaphore, the arm is bent to a parabolic curve, in plan, and has on its face a strip of glass with its surface roughened to catch and reflect the light from the sun by day and the light of the lamp by night. The arm is painted red, so that by day it resembles an ordinary semaphore, except that it is very bright. The glass side of the signal lantern, facing towards the arm, has the upper half red and the lower half white, so that at the "danger" position the engine driver sees a horizontal beam of red light, while at the "track clear" position he sees a downward line of white light, as reflected on the arm. This signal, manufactured by the National Switch and Signal Company, has been in service on several roads and has given satisfactory results. The Union Switch and Signal Company has an illuminated signal consisting of a hollow "box" arm having a glass slip in the side and reflectors to throw the light into the "box" from a lantern near the pivot of the arm; colored glasses moving in front of the lantern, in accordance with the movement of the arm, cause a change of color of the reflected light for different positions. On the Boston and Albany Railway and the Old Colony Railway, systems are in use by which two lights in a horizontal line are shown when the arm is horizontal or at "danger," and two lights in a vertical line when the arm is dropped to the "track clear" position; the lights are of different colors.

Home and distant and starting signals should be placed at stations, to protect trains standing at the platforms. At yards and side-tracks all switches should be fitted with clearly distinguishable ground signals to facilitate the operations of switching; and there should also be the regular home and distant signals to protect the entrance to the yard or side-track. Signals should also be used to protect dangerous places, such as sharp curves in deep cuts, as accidents are continually occurring at such places. Automatic signals would be most suited for these locations.

Railway grade crossings should be thoroughly protected by signals, and should also have a derailing switch and a blind side-track for trains which run past the danger signal. It may not, however, always be practicable to use a derailing switch, as for instance, where there are more than two tracks it might cause a crossing collision. On the Le-

Digitized by Google

high Valley Railway a torpedo-placing signal, operated in connection with the semaphore, is used to obviate the use of a derailing switch. This torpedo signal is one of the National Switch and Signal Company's specialties. It is, however, only a precaution and not a preventive. A device which promises well for such cases, but which has only been tried as an experiment on the Old Colony Railway, is the Rowell automatic stop. It consists of an iron plate forming an inclined plane at the side of the rail; on the engine a pipe connected with the train brake is laid to the front end, where a cock is placed with a pendent rod hanging from its handle; when the signal arm is at "danger" the inclined plane is thrown into position, and by throwing up the rod on the engine opens the cock and applies the brakes. The necessary equipment is simple and inexpensive. This device may be used separately as a portable signal to prevent accidents in case of a breakdown, repairs to track, etc. Another device consists of a movable rod on the signal post, which projects over the track and strikes a catch or breaks a glass tube on the engine, setting the brakes. Derailing switches should also be used at drawbridges. Highway grade crossings should be protected by visible or audible signals in addition to the usual signboards, but few, if any, such signals have yet given any reasonable degree of certainty in action; the signal failing at some critical time, or by its uncertainty leading to its being disregarded. Without actual tria , however, no practically reliable apparatus can be expected, and trials of different devices are now in progress on some roads. The Board of Railroad Commissioners of the State of New York, in their report for 1885, under the head of "Safety Appliances," referred to this matter, and gave the following sound advice:

"The occasional failure, from whatever cause, be it neglect, malicious interference or inherent defect, seriously impairs its usefulness as a sole warning. It is like the man who is known to not always tell the truth. The periodic lie suffices to throw suspicion on every statement he makes. As a general proposition, and after watching the results with much care, the Board is satisfied that automatic apparatus, whether electric or mechanical, should serve only to supplement human watchfulness, not to supersede it. In the former capacity it is invaluable, serving to keep men alert and to remind them of their duties; in the latter it is more dangerous even than unaided human care with all its proverbial weakness and fallibility."

Audible signals are sometimes used in connection with visible signals, as the torpedo machine already referred to, in which the operation of setting the signal at "danger" causes an arm having a torpedo at its

Digitized by Google

extremity to project over the rail. Arrangements for automatically sounding the engine whistle have also been devised.

Within a comparatively recent date electricity has come into practical use as a valuable adjunct to signaling operations, and experiments are being made to further extend the field for the application of this power. It is sometimes claimed that it is too uncertain in action, but I think the science of practical electricity has reached such a degree of perfection that it can be relied upon for certain work. It is largely used in connection with switch and signal interlocking apparatus, in some cases for very extensive plant, and is also in use for automatic block and other signals. Various systems of electrical signaling, more or less complicated, have been devised, some of which are intended to be absolutely or nearly automatic, and in some of which it is proposed to use electricity not only to operate the ordinary signals but also to supplement them by visible or audible signals in the engine cab or on the train. While it is probable that the use of electricity in signaling will be considerably extended, the remarks of the Railway Commissioners, quoted above, are decidedly pertinent in connection with the proposed comprehensive automatic systems. The Sykes system of electro-mechanical block signaling, in which the art of signaling is brought to a high degree of elaboration and perfection, is in use on the New York Central and Hudson River Railway, the New York, New Haven and Hartford Railway, and the New York, Lake Erie and Western Railway.

Compressed air is also successfully applied to complicated signal operations; this system is a specialty of the Union Switch and Signal Company. A compressor is located at the signal cabin and the air conveyed in pipes to and up the signal posts to the apparatus which operates the semaphore arm; electricity is used in connection with this apparatus. The pneumatic system, as well as the electrical system, of signaling, includes the interlocking of switches and signals.

The Old Colony Railway uses special electric signals at some road crossings, including two highways at Brockton, Mass., these streets being occupied by an electric street railway. These signals display a red target when in the normal position, and a white target only when cleared by the closing of the electric currents by the gatemen. This road has illuminated arms for some of the switch signals, the line of light showing the position of the arm. Very complete signal equipment is in use to insure efficiency and safety in operating the heavy traffic.

Digitized by Google

Interlocking apparatus and derailing switches are also used at several junctions and other points. The distant signals are located 1 000 to 1 500 feet from the home signals. On some sections the automatic electric block signal system is in use.

At sharp curves on city elevated railways signals are sometimes used, operated automatically or by a signal man, to indicate if the track is clear. An automatic block signal system, specially adapted for city elevated and suburban railways, has been designed by Mr. Black, Road-master of the Manhattan Railway (Elevated) of New York. Signal posts, with a signal arm or target on each, are placed at intervals along the track, and are operated by track levers moved by the wheels of trains. A train passing one of these signals sets it at "danger," and at the same time releases the last signal behind. The signal thus set will remain at "danger" until automatically released by the train passing the next signal ahead. This system is in use on elevated railways in New York and Brooklyn, and on the Suburban Rapid Transit Railway of Staten Island.

The interlocking of switches and signals is one of the most important advances in the improvement of railway track equipment, and is necessary in order to obtain the full efficiency of the signal system in insuring safety and rapidity in the operation of heavy traffic. This class of apparatus has been brought to a high degree of perfection, and it has been developed into very comprehensive working. Explanations and diagrams tend somewhat to convey the impression that the apparatus is extremely complicated, but the successful operation under severe conditions of service for a long period of time has proved its practical value. The subject of interlocking is referred to by the Railway Commissioners of some States in their annual reports. In Europe very complete systems of interlocking are adopted, with a very complete signal equipment, owing to the great amount of traffic to be handled. In Australia, the system is widely used on some of the government lines, as will be seen by the following extract from a report by the United States Consul at Sydney, New South Wales:

"The semaphore system of signaling is principally used, every station and junction having home and distant signals. Interlocking systems, similar to those in use in England, are adopted at all stations on the main suburban lines and on the principal country lines. A special arrangement for interlocking is employed at sidings where no one is in charge. When the sidings are not in use, the main line signals indicate

Digitized by Google

that the track is clear and the levers of the apparatus are locked up so that they cannot be tampered with. Conductors requiring to switch onto the side track are provided with a key to unlock the apparatus, and they must then fix all necessary signals before they can open the switch. When the switching is finished the conductor must close and fasten the switch before he can lower the main line signals to show 'track clear,' and these signals must be lowered before the key can be taken out of the lock of the case which covers the levers. At small sidings, which can conveniently be switched by trains carrying the time staff, fixed signals are used. All sidings in direct communication with the main track are provided with either stop blocks or safety switches, to prevent cars from being moved or blown along, so as to foul the main track."

In all apparatus for the operation and interlocking of switches and signals, simplicity and uniformity of parts should be aimed at. It is not practicable within the limits of this paper to go into details as to the different systems mentioned, as such details would contain matter for an elaborate paper in itself. The subject is one which has not received sufficient attention in this country, but should now be more carefully considered as the traffic of the railways increases. Where adopted in this country, successful results have attended their use at points involving the operation of a large number of switches and signals.

To persons engaged or interested in signaling or kindred matters, I would certainly recommend the perusal of the books published by the leading manufacturers of such plant, as many of these books contain clear and carefully written descriptions of the different apparatus, their uses and advantages, forming text books on this important subject. I would also refer to a paper by Mr. R. H. Soule, on "Railway Signals and Signaling," read before the New England Railway Club and published in *Engineering News*, New York, December 21st and 28th, 1889.

Bridge Tracks.

The best plan for carrying railway track over bridges is to build the bridges with solid floors and lay ballast over them in the ordinary way, making a continuous roadbed. This system is now being introduced on the New York Central and Hudson River Railway, and Mr. George H. Thomson, M. Am. Soc. C. E., Engineer of Bridges of this road, has the honor of being the pioneer in introducing this important improvement in the United States. The dead weight of the ballast and floor not only requires a heavier construction of the bridge, which in itself is an ad-

Digitized by Google

vantage, as tending to diminish the bad effects of the vibration caused by trains, but the ballast also serves as a cushion to absorb the motion which causes this vibration, and in this way materially diminishes the destructive effect upon substructure and superstructure. The roadbed on the abutments and for about 50 to 100 feet back from the bridge should have a depth of about 2 feet of ballast under the ties, so as to give an elastic roadbed nearly equal to that of the bridge, in summer and winter. In this way the axles of engines and cars will not be subject to the strains due to passing abruptly from a track on a hard frozen bank and a rigid abutment onto that on a comparatively elastic bridge floor, and *vice versa*. Of course, the cost of the bridge is increased, but it is false economy to try to save a little in the first cost of a bridge; and the increased safety of the track and the greater durability of the structure should far outweigh any such considerations of seeming economy. The floor of a bridge on the New York Central and Hudson River Railway, which may be taken to represent this system, has a series of transverse troughs, built up of plates and angle irons, the troughs being open at top and bottom alternately, forming a "corrugated" floor. These troughs are 16 inches wide and 18 inches deep; the ballast is filled into the alternate ones and 8 x 8-inch ties are bedded in it, leaving about 12 inches of ballast under the tie. Wooden blocks placed between the front side of the tie and the side of the trough prevent the creeping of the track. In other cases, rolled iron troughs, with flaring sides, are used; the sides overlapping, and being secured by a row of rivets; with these the floor is covered with ballast, in which the ties are placed. For short spans, the built-up troughs or rolled troughs may be placed longitudinally, and in such case the latter are found the more economical. (See *Engineering News*, New York, March 31, November 16 and November 23, 1889, for further data respecting these bridge floors.) The arrangement of the floor can be varied to suit different cases.

Solid iron floors for railway bridges are almost universally adopted in England, generally with an ordinary road-bed of ballast and cross-ties, but sometimes with wooden stringers under the rails and no ballast. Buckle plates are there very generally used for the floors. Different forms of longitudinal or transverse troughs are also manufactured for bridge floors. Sir A. M. Rendel, M. Inst. C. E., Consulting Engineer for the State Railways of India, has recommended the use of corrugated iron floors covered with ballast, as being stronger and more economical

Digitized by Google

than the ordinary plank floor, while it increases the structural strength of the bridge.

The engineer, however, has often to do the best he can with existing structures and with a small amount of money. Where a railway track is laid over a trestle or a bridge with the ordinary open floor system in general use in this country, it should be well protected by guard rails, preferably large timbers faced with iron, well bolted down and braced. The ties should be placed close together, with spacing blocks between them and should be well fastened down, so that in case of derailment the ties will not be bunched together and leave gaps for the wheels or trucks to drop into. When at Sault Ste. Marie, just after the convention of 1888, I took occasion to notice the guard rails on the long bridge over the river. The guard rails proper are 6 x 9-inch timbers, laid flat, boxed out 1 inch for the ties and faced with an angle iron; the guard rails are bolted to the ties on the outside of the track, and are 8 inches clear from the track rails. Along the ends of the ties are 9 x 9-inch timbers, also boxed out for the ties. The guard rails flare out on the abutments, but the outer timbers end at the bridge portals. The Starrucca Viaduct, which is a long, high masonry viaduct on the New York, Lake Erie and Western Railway, and which has no parapet wall above the track, has inside guard rails of ordinary rails. Similar guard rails are used elsewhere, and in some cases, as on the New York and New England Railway, they are extended some distance along the approaches. The Michigan Central Railway uses three parallel lines of ordinary rails between the track rails. The Boston and Lowell Railway has its track at culverts protected by outside guard rails of ordinary rails with the ends flaring out from the track.

All floors at openings, such as at bridges, trestles, culverts, etc., should be strong enough to carry a derailed truck, and should be fitted with guard rails sufficiently strong to keep a derailed truck on the structure. Rerailing appliances should be placed on the approaches of all such structures.

Tracklaying and Maintenance.

In order to obtain and maintain a proper standard condition of track, care is necessary in all tracklaying and maintenance work. Special pains should be taken to see that work is properly and carefully done and finished up in laying new or heavier rails, or in putting in and fixing any new and improved forms of joints, fastenings, frogs, switches, or other

Digitized by Google

track material or appliances, so that a fair trial may be made, enabling comparisons to be drawn between different forms of track and appliances, and so that the best results may be obtained. Similar care should be taken in maintenance, and observations should be made of the behavior of any new devices under trial. Careless work on such improvements in track may negative the advantages and economies that might and should properly be derived from their use.

For track laid with heavy rails, and, in fact, any first-class track, which is to be raised, during maintenance, the use of track jacks will enable better and quicker work to be done than by any other means. Jacks are used in Europe for work on lines with 100-pound rails, and greatly facilitate and improve the track work.

Track laying by machinery is, of course, only for rapid construction, and while track laying machines have been successfully used for such work, yet this class of track laying has no relation to the standard of track or improvements in track to which this paper refers.

Inspection.

Systematic and careful inspection, with reports thereon, will aid very much in keeping up the standard of the condition of the track, and the premium system adopted on some roads gives an incentive to the track staff which is probably more effectual than mere praise or censure. When a new type of track, or new track appliances are being tried, especially if the trial is with a view to adoption, and not merely complimentary, care should be taken to see that the conditions are favorable, to observe the behavior of the new work, and to notice if the new device does the work it is claimed and intended to do, or if it requires special attention to keep it in order, etc.

In addition to the regular inspection by the officers of the road, a number of the leading railway companies have adopted the practice of having the track gone over once or twice a year by the track inspection and dynagraph car, designed and operated by Mr. P. H. Dudley. This car, with its registering apparatus, is one of the most interesting and valuable machines ever designed in connection with railway work, but its construction is too intricate and its work too detailed to be more than briefly referred to in this paper. The car is hauled over the track at a moderate speed, about 15 to 18 miles per hour, and its machinery registers, on a continuous traveling roll of paper, the state of the rails, the

Digitized by Google

condition of the joints, and other matters, while low joints and other defects are marked by a jet of paint thrown on the rail, so that the foremen of the section gangs can see at once where their work is required. This inspection and the report compiled from the lines traced by the machine assist greatly in enabling the companies' officials to decide upon the necessary work to be done in repairs, renewals, new track, etc., and also in enabling the section gangs to put the track in proper condition.

Reports and Accounts.

Reports on the construction and condition of the tracks of railways are of great interest and value, but are not generally easy to obtain. Some of the Railway Commissioners' reports, in the statements of inspection or in the returns furnished by the companies, give particulars of the track, and these particulars are very valuable for information and reference, as a matter of record. These State Commissioners, however, might with advantage extend this branch of their inquiries and reports, and the National Railway Commission of the future, whose formation has already been suggested in this paper (see "Government Regulation of Railways"), will naturally make a special feature of collecting, compiling and preserving statistics of this kind.

In construction and in maintenance, careful notes should be made and records kept and tabulated, to show the work done, time occupied, tools and materials used, items of cost, etc. In his report to the Louisville and Nashville Railway Company in 1875, Mr. Albert Fink, M. Am. Soc. C. E., then Vice-President and General Superintendent of the road, recommended the adoption of a complete, comprehensive and uniform system of railway accounting. The report contains, also, some very interesting matter on the subject of track work. The accounts of this road were kept at that time on the system recommended, and the department of "Operating Expenses" gave details of the cost of track work, maintenance and repairs, prepared from the records of the various officers. (See Appendix F.) The half-yearly reports of the East Indian Railway Company, in India, also contain much statistical matter in relation to the track and track work. All work of this kind, which puts things on record, and enables comparisons to be made, and the relations between work, expenditure and results to be clearly comprehended, tends toward the improvement of work and methods.

Digitized by Google

Instructions to Trackmen.

The work of the men in actual charge of the track—the roadmasters, section bosses and foremen—carries a considerable responsibility, and the most thorough means should be taken to keep them fully informed as to their duties, and to impress upon them that the safe and proper condition of the track is the first and most important consideration. An excellent plan is to issue "instruction books," similar to those already in use on some roads, but which should be more extensively adopted. For a road starting to adopt this plan, the books should at first be small pamphlets, so as not to incur much expense, and after they have been in service for a sufficient time to become well known and well used, opinions and suggestions should be invited from time to time from the men on the road and from officers of other roads in order that improvements may be made by eliminations, alterations and additions, so as to embody the best results of actual experience. A handy pocket book can then be made up for permanent use. The matter should be clearly and concisely written (care being taken to avoid ambiguous phraseology), so as to be readily understood, and the use of diagrams will add to the usefulness of the book. It should be clearly printed on strong paper, and strongly bound. The illustrations should preferably be on the pages of the book, or at any rate on sheets not folded more than once or twice, as large folding sheets are awkward to handle and are liable to be very soon torn.

Special instructions should be given to the section bosses when any important improvements are to be carried out or new devices tested.

Street Railway Track.

As supplementary to railway track this subject is worthy of brief consideration, especially as considerable improvements have been made of late years by the introduction of metal substructure. The old and almost universal system of track with flat rails of side-bearing or center-bearing section, spiked to wooden stringers which are laid on wooden cross-ties, is a very objectionable system for city streets; the rails have little vertical stiffness, and under heavy street traffic the stringers are crushed and the spikes loosened, so that the rails work up and down with every passing load, and in wet weather spurt the mud in all directions.

Digitized by Google

The following paragraph from an issue of the New York *Times* in November, 1889, is a curious commentary upon this state of affairs, and the Judge's decision is good enough to put on record as amusing if not instructive:

"*A Bad Habit of Car Rails.*

"Alexander Taylor, an old resident of Yonkers, was walking across the tracks of the Second Avenue Railroad Company at the corner of Fulton and West streets, this city, on the 26th of January last, when a passing car caused a rail to raise up at the end suddenly so that it tripped him, and, being seventy-two years of age, he fell heavily and received severe injuries. He brought suit for $10 000 damages against the company, and it came up before Justice Dykman, at White Plains, yesterday. The Judge, after hearing the complaint, dismissed the suit, holding that the rail had been properly spiked and jumped up, just as any rail with a car at the other end of it naturally will."

The joints, too, are generally bad, the rail ends resting on an iron plate and spiked in the usual way. The stringers and ties are fastened together by iron angles or knees, with spikes in both timbers. Cable railway tracks are necessarily of heavier and better construction than the ordinary horse railway tracks, metal and concrete being extensively used. With the introduction of mechanical motors of increased weight, improvements will be necessary in the older tracks.

In Europe iron and steel track on a concrete foundation is the approved form of construction, and is probably the track of the future for the cities of this country; but until the streets have proper foundations, and are properly paved and maintained, railway companies will not care to expend large sums of money on an improved track, the advantages and economies of which would be nullified by the condition of the streets. For small towns and for lines with a limited amount of street or car traffic, the timber substructure may be admissible for some time yet. The rails used with the metal substructure are of three general types of sections: 1, the girder rail, which is an adaptation of the ordinary railway flange rail, and is laid directly on the concrete foundation or on metal longitudinals or cross-ties bedded in the concrete; 2, the flangeless T rail, the web of which rests in iron chairs; and 3, the saddle rail, which is also carried on iron chairs placed at intervals. All of these sections have in various systems been adapted to American practice. In Europe the rails are almost invariably grooved. So far the grooved rail has not met with much favor in this country, and it has not been much used except for curves. One of the principal reasons for this is that this form of rail is advantageous only

Digitized by Google

with a good pavement, and would effect little or no improvement where deep ruts are allowed to exist outside the rail, which is the present condition of the average city street. It is claimed that the groove would clog up and would require frequent cleaning out, which would involve an additional expense. When the time comes, however, that the main streets, at least, of important cities are properly constructed and maintained, then the use of the grooved rail should be enforced by municipal ordinance, and it will probably be found then that there is little more trouble than in other countries where these rails are used, while any expense for cleaning the groove would be offset by the reduced cost of maintenance. New York City has taken a step in this direction, the Department of Public Works having approved a form of grooved rail which is required to be used on new lines and in the rebuilding of old lines. One trouble experienced with the English system of track is that the street traffic, keeping to the track as far as possible, tends to wear ruts in the paving along the side of the head; to prevent this it was proposed in one case to set thin iron plates in the joints between the paving blocks next to the rails. On the Beacon street line of the West End Street Railway, Boston, Mass., a heavy grooved head girder rail is used, which was designed by Mr. Thomas Doane, M. Am. Soc. C. E., and was made in England to his plans. It resembles the English type of girder rail. It is 7 inches high, with a bottom flange 6 inches wide, and a head 3¾ inches wide; it weighs 99$\frac{7}{10}$ pounds per yard. Mr. Doane states that there is a difference of opinion as to its success. The only objection is that there is a liability of the groove to get filled up with dirt, and it costs something, though little, to keep it free. This particular rail is, perhaps, unnecessarily heavy for horse or cable cars, which are light loads, but now that the West End Street Railway Company is running electric motors, which are much heavier, it is found that the ordinary tracks are not strong enough, while this girder rail will carry the traffic very well. It offers no resistance to the crossing of carriage or wagon wheels, and remained in good line and surface after nearly two years' service.

The Providence girder rail track, manufactured by Wharton & Co., of Philadelphia, and Keefer & Co., of Kansas City, consists of a rail about 7 inches high, with a narrow base keyed in a dovetailed groove in cast-iron chairs, 18 to 24 inches long, which rest on a concrete foundation, and are placed about 7 feet 6 inches apart. The head is flat, 2

Digitized by Google

inches wide; the paving between the rails is $\frac{1}{8}$-inch below the top of the rail, to allow room for the wheel flanges, while the paving on the outside of the track is flush with the top of the rail. Tie-rods are used to hold the rails to gauge; they are of $2\frac{1}{2}$ x $\frac{1}{2}$-inch iron, threaded at each end, with a nut on each side of the web of each rail, which prevents any lateral movement. This rail has been used for cable roads by making provision for keying the rail into seats formed on the yokes.

The Wharton Company, in addition to the foregoing track, has other special systems. One of these, now extensively used, consists of girder rails or flangeless T rails, the head being of side-bearing on center-bearing section. With the latter rail the web and outer side of the head rest on iron chairs, placed on wooden cross-ties. With the girder rail the flange rests on a broad seat of the chair.

The Johnson Company, of Johnstown., Pa., makes a specialty of girder rails and flangeless T rails with heads of different forms, and carried in different styles of chairs. For use with the prevailing system of paving, the company makes a rail with a groove on each side of the head, and another grooved rail with the guard lower than the tread. The joints of the girder rails are spliced by a pair of bars of channel section (] [) with four bolts. Girder rail crossings, with filled grooves, and metal cross-ties for street railways are other specialties.

The Metallic Railway Supply Company, of Albany, N. Y., supplies a track with hollow cast-iron stringers; these are filled with sand, and a flangeless T rail, with side-bearing or center-bearing head, is laid, the web lying inside the stringer and the head resting on the top. Flat tie-rods are fitted into holes in the side of the stringer and the web of the rail, and are secured by cotters or keys, so that vertical motion is prevented. This track has been in service for about four years, and has given good results. Cast-iron is not generally considered a good material for railway track, but it must be remembered that the conditions of a track for street railway cars and a track for heavy locomotives and trains running at high speed are under very different conditions. Breakages have been very few.

All these companies manufacture numerous forms and modifications of metal track. A rail has been designed by R. T. White, of Boston, which is a compromise between the American side-bearing rail and the English grooved rail, the lower flange of the former being inclined upward from the middle so as to form a wide flaring groove. It is

Digitized by Google

intended for city lines, and is said to be more easily crossed than the ordinary rail. This rail is of saddle section, and rests on iron chairs, to which it is secured by horizontal bolts passing through the sides of the rail and the top of the chair.

A very substantial track was designed by Mr. Edward Wilson, M. Inst. C. E., for the tramways at Southampton, England. The rail is of steel, of the flangeless T section, with a grooved head, and weighing 55 pounds per yard. It is 3½ inches wide and 3⅞ inches deep; the groove is 1½-inch wide at top, ¾-inch at the bottom, and ½-inch deep; the top of the head is not horizontal, but slopes slightly downward from the edge of the groove to the side of the head, so as to allow for wear. The web of the rail rests in a groove in cast-iron chairs with a flat base of 12 x 16 inches; the upper part of the chair is 8 inches long (11 inches for the joint chairs) and the same width as the rail head. The chairs rest on a concrete foundation and are connected by flat tie bars. The rails are secured to each chair by two taper steel cotters driven horizontally through holes in the chair and the web of the rail; at the joint chairs there are two cotters to the end of each rail. This fastening is very secure, as it prevents any vertical motion of the rail, and the paving prevents the cotters from slacking back.

Improved switches, operated by the horses or the car, have been very generally adopted in this country, but the frogs and crossings are usually of a very crude type, hard on the horses and the cars, and unpleasant for the passengers. In many cases one rail is merely cut at the intersection, and a gap left at which the wheels batter the rail ends in a barbarous manner, while at such crossings the rails are often very loose. A track crossing designed to give a smooth and easy passage to the cars was brought to my notice a few years ago, the principal feature of which was the gradual elevation of the lower flange of the head of center bearing and side bearing rails or the bottom of the groove of grooved rails, so as to just reach the wheel flange, the head of the rail being depressed so that the wheels ran across on the flanges. While excellent in theory, and working well in miniature, yet in practice the wear of the crossing by heavy wagons, etc., and the wear and different depths of the car wheel flanges, would probably soon have impaired its efficiency in service. With good street pavements and good street railway tracks, some improved form of crossing will certainly be a necessity.

Digitized by Google

Conclusion.

The object of this paper has been to show to some extent the improvements which have been made and to suggest some further improvements which might be made in railway track and in matters relating to track, and I think that it will show, on the whole, that considerable improvements have been made and are still in progress, in spite of the main trouble of want of money, or difficulty in getting the money directed into channels leading to track work. A considerable amount of scattered information has been brought together, but in describing the various improvements and appliances I have not attempted any complete list or description, but have intended to indicate the way in which the improvements are being made, and I beg to express my thanks for the information kindly furnished from various sources. It is my hope that this paper may prove not merely interesting but of some use to those in whose hands lies the important matter of the responsibility for the track of our railways, and that it may thus aid in "the improvement of railway and street railway track."

APPENDIX A.

[Sandberg's New Goliath Rail with Steel Tie-Plate.]

By Mr. C. P. Sandberg, A. M. I. C. E.

There may not seem at present to be any urgent call for a new Goliath rail, inasmuch as the first type was constructed only three years ago, and therefore our experience of it has, as yet, been comparatively small. During that period, however, several important facts have been brought to light with respect to rail sections generally, which also affect the Goliath, and as this is a costly rail it had better be made right at once. These facts are:

First.—That the big rail head, owing to the fast rolling in the modern mills, does not seem to wear well, inasmuch as the physical hardness is only in the surface and not in the interior of the rail head. But we shall not be able to make rails chemically hard with 0.50 to 0.60 per cent. of carbon, as is now used in American mills, until we, in Europe, adopt the American mode of curving the rail while hot, so as not to spoil the strength of the rail by cold straightening presses and gag marks on the flange, which might cause fractures and accidents. It will anyhow be preferable to have a wider rail head than a thicker one. The great value of this modification will be seen not only in the longer

Digitized by Google

duration of the rail, but also in the increased life of the tire, which would not require to be turned if the rail head were as wide as the bearing surface, or nearly so. It would also add to the tractive power of the engine by offering a greater surface to the driving wheels on the rail; in fact the tractive power of an engine on the wider rail head would be greater than that of an engine of equal weight on the narrower one.

Second.—It would offer a better bearing for the fish joint without sloping the sides of the rail head, as has been done so largely in America, but has now been abandoned on account of the side friction to the tire flange.

Third.—The crushing load on the driving wheels would not exceed the limit of elasticity of the steel in the wider rail head.

These seem to be facts of very great importance, and well worthy of attention. The new Goliath rail section of 1889 has, therefore, been designed with a head of 3 inches width instead of $2\frac{3}{8}$ inches, and $1\frac{3}{4}$ inches depth, instead of $1\frac{7}{8}$ inches; thus giving a thinner head with greater wearing surface in width instead of in depth, and giving 1 inch bearing of rail head against the fish instead of $\frac{3}{4}$ inch.

The distribution of weight in the new Goliath, as compared with the old, is as follows:

	1886.	1889.
Head	43.5	45.5
Web	23.9	22.0
Flange	32.6	32.5
	100.0	100.0

Regarding the top radius, in respect of which there has been so much discussion in America with the view of not loading beyond the limit of elasticity, it has been made 8 inches instead of 10 inches; it is thus made rather more round, for the reason that the tires on old lines, where such a Goliath rail would be used, are certainly not flat, but more or less worn hollow. By an increase of width of $\frac{3}{8}$ inch or $\frac{1}{2}$ inch in the old rail head the tire would, on curves, be brought to bear on the outside part, and thus lessen the acting depth of the tire flange by as much as it is worn hollow, so as to cause some risk of running off the rail, and this would be the greater the more the rail head is flat and the tire worn hollow.

Therefore the 8 inches radius of rail top, which is supposed to also represent the radius of the existing wearing surface of the tire, has more chance of safety by keeping the tire on the rail, with the application of a Goliath rail of 3 inches width of head, on a road where rails with $2\frac{1}{2}$-inch heads have been used. As the new Goliath rail will offer a wider bearing for the tire it will practically come to the same thing as if the rail head were flat and narrower, namely, the prevention of loading beyond the limit of elasticity, particularly as the wider surface in the

Digitized by Google

skin will be harder through colder rolling than in the thick narrow head. The ½-inch corner radius of rail head, as adopted in America, would not be applicable in Europe until the bogie system is generally adopted here as there, therefore the ½-inch radius is adhered to. The vertical side is maintained as a medium between the American rail sloping inward to the top, and the Russian and German rails sloping outward to the top. The lower corner radius of rail head may be as sharp as it can be rolled, say $\frac{1}{8\frac{1}{2}}$-inch. The bearing of the fish, 1 inch, against the rail head should be perfectly straight and so should the fish also be, as if planed in a planing machine. This explains the changes introduced into the rail head. Now as to the flange of the new Goliath. The weak point of the road with flange rail generally as compared with the bull-head rail in chairs is the insufficient width of the base and the weak method of fixing the same to wooden ties. The width of the flange of the Goliath design of 1886, only 5½ inches, is not sufficient for soft pine ties, as compared with the 12 to 15 inches which the cast-iron chair affords for the bull-head rail section. It would, therefore, be better to give up the plan of laying the rail flange direct on the tie, and to adopt a steel tie-plate of sufficient surface, or equal to that of the cast-iron chair, nearly 100 square inches, say 12 x 8 inches, and ½-inch thick, so as not to bend it, and fasten the same to the tie with three fang-bolts, wood screws or spikes, using clips and steel keys for fixing the rail to the base plate, thus enabling the rail to be made of harder steel, and affording a firm attachment of the rail to the tie, while by knocking out the steel keys it offers a quick means of changing either rail or tie when one of them becomes defective. The insufficient method of fastening the flange rail direct to the wooden tie by spikes or wood screws would thus be done away with, and a longer duration of the tie would be insured. It should be observed that the rail flange has on both sides a bearing of the whole width of base plate of 3 inches, as against the old method with a bearing against a spike or a wood screw, and that the widening of gauge on curves could be arranged by using steel keys of different sizes, as with metal ties, of which several hundred thousand tons have now been laid in different countries with satisfactory results. Should the tilt of the rail of 1 in 20 be needed, this could be given by rolling the base plate of varying thickness under the rail flange. The fixing of the base plate to the tie is effected by means of spikes, wood screws or fang-bolts. The tie-plate gives the tie of soft wood a bearing surface of more than double that given when the rail is laid direct upon the wood, which will naturally double the life of the tie. The extra cost or additional weight of 13 pounds to the base plate would be covered by the saving thus effected, and the total expense of the Goliath rail track would still be less than that of the English track with bull-headed rails and cast-iron chairs. If this plate is taken at $30 per ton it would still cost less than the cast-iron chair, which weighs 40 to 50 pounds, and costs $12.50

Digitized by Google

to $15 per ton. By this means it might be possible to obtain an equally good and safe track with flange rails as with the English bull-head type of rail, and at a somewhat less cost. But, as is now only too often the case on the Continent, to employ half the weight of metal in the flange rail as in the bull-head section (with chair included), and yet to demand equal strength, is to ask too much of the flange rail section. An interesting book called "Express Trains, English and Foreign," by Foxwell and Farrer, just published, classes as "English express trains" all trains run at 40 miles an hour and over, including stops, and as "Continental express trains" those running at 29 miles an hour or over. Statistics show that in England only half the number of passengers per cent. are killed as compared with the number on the Continent. These facts speak volumes for the superiority of the English railways and the need of improvements on the continental roads. In the second edition of this valuable work it would be interesting if the disproportion between weights of rail and weight of engines were given in a column for each express, so as to show where an increase of rail weight in the direction of the Goliath is mostly wanted. As it would be useless to try and introduce the English type of track in those countries where flange rails are already adopted, the only way to obtain an equally safe and strong road for heavy traffic as on the English lines, seems to be to strengthen those by the adoption of the Goliath rail with steel tie-plates, but to let every one keep to his system of rail. And, fortunately, the price of steel has become so low that it would now become an economic advantage by the saving in the cost of maintenance; the public safety, with increased speed, would be an additional advantage. To take a striking illustration one need only compare the rail weight a :d the rolling stock with guns and armor plates. The guns have been gradually increased to the weight of 100 tons and the armor plates similarly increased in thickness in equal proportion at the same time, say up to 18 inches. The rolling stock has doubled in weight and the speed been increased, but the rail has been left the same, especially the flange rail on the Continent. The rail, too, has something to do with war and defence, for just as the armor plate must withstand the battering of the guns, so must the rail withstand the battering of the rolling stock and be increased in proportion to the weight of the latter and at the same time. But this is precisely what has not been done, for while the locomotive department has increased the rolling stock as a matter of necessity to take the increased traffic, the track department seems to have had no influence with the directors, and so the disproportion between the track and the rolling stock has come to be an established fact on many continental roads with heavy traffic.

The weak rail may, by stopping and patching, keep the road open under ordinary conditions, but in time of war, for instance, when the traffic is doubled or even trebled, where would the weak rail be? And

Digitized by Google

what could be done to it when there is no time for patching and for repairs? The strong rail has, in truth, a great deal to do with the defence of a country, for the army which can, with provisions and ammunition reach the battle field first, stands the best chance. Experience has sufficiently proved this already in 1870. Thus the Goliath rail is really a means of defence, at least in Europe. Besides, enough has been said to prove its *raison d'etre* even in time of peace. The recent experience regarding change of rail section as stated above, will, it is hoped, be considered to justify the appearance of this second edition of Goliath rail section, and as the design of 1886, after much persuasion, broke the ice by its adoption on the Belgian State Railways, it may be hoped that that of 1889 will further the general adoption of this rail more rapidly, not only by offering a better section and a stronger joint, but also by a stronger and firmer fastening to the wooden ties and the longer duration of the same by the use of steel tie plates.

At the Paris Exhibition there was a sufficient display of Goliath engines and carriages, but very little was exhibited to illustrate the proportionate strengthening of the road; in fact almost the only exhibit of this kind was a series of rail models called "Goliath rails" in the Machinery Hall (Class 61, Railway Material), showing the history of the Goliath rail: first the drawing of Sandberg's Goliath of 1886; then the model of the same, 1887, afterwards the section of the Goliath adopted for the Belgian State Railways (now 15 000 tons), as made by the Société Cockerill, of Seraing; and lastly, drawings and models of Sandberg's Goliath of 1889, with the addition of the large steel tie-plate with clips and steel keys for fastening the rail, and flange bolts for fastening it to the wooden ties.

It may naturally happen that in its adoption modifications may be continually introduced by different engineers, from individual experience and local conditions, such as climate, etc. This, however, does not affect the principle now proposed to improve the continental roads, which is free for adoption by any one, and it may be added that the mode suggested is based on long experience and is at least free from private interests of any kind whatever.

The new Goliath rail has, in addition to a wider head, been modified by means of a narrower flange with a view to the use of steel tie-plates. The weakest part of a road using flange rails is the inefficient fastening of the rail to the tie, as well as the short life of the tie itself, the rails outlasting several ties since steel has come into use. Every year the wood used for ties is becoming of softer texture, and consequently does not last so long as formerly, and the frequent renewal of ties now forms the largest item in the maintenance of track, particularly where flange rails are laid directly upon the ties; the bearing surface being much too small in any section that can be easily rolled to prevent the rail cutting into the wood. The width of the rail flange varies from 3½ to 4½ inches

Digitized by Google

for a 50 to a 70-pound section, up to 5 inches for the 100-pound Goliath. Rails have been made with extra wide flanges, such as 6 inches and even 6½ inches, but with great difficulty and at an increased cost. With an 8-inch flat surface of tie this would give a bearing of from 28 to 40 square inches; but wooden ties are often cut semi-circular, the flat surface being placed down in the ballast, thus reducing the actual bearing surface to about one-half. The increased weight of rolling stock and the higher speed soon drive the flange into the tie, prematurely destroying it, and interfering with the gauge of the road. Oak ties, of course, will have the longest life, and fir ties, creosoted, will last only seven to eight years; therefore the introduction of steel ties wherever wood is dear has become a radical remedy of great saving and economy in many countries; but there are circumstances where the cost of steel ties will not balance with that of wood, and where the increase of duration of the latter through an extended bearing surface would be desirable. As such increase cannot be obtained through the increased width of the rail flange to anything like the extent wanted, the introduction of tie-plates seems inevitable, if roads using flange rails are to equal in safety and economy those in England using bull-headed rails and cast-iron chairs. But these chairs have about 100 square inches bearing surface on the tie, and under these conditions the tie lasts nearly twice as long as when the rail is laid directly upon it. The chair is fixed by large fang bolts of about ⅞ to ¾-inch diameter. Therefore the tie-plate should have these two conditions: as large a bearing surface and as firm a fastening to the tie as the chair, in order to obtain equal results.

With this view a steel tie-plate has been designed, 7 x 16 inches by ½-inch thick, giving a bearing surface of 112 inches, and having the same method of fastening as the chair. Both conditions, of course, may be modified according to circumstances, such as the size of ties most generally obtained, climatic influence, and each engineer's fancy for fastenings.

Seeing that the rail can only give from 30 to 40 square inches bearing, the treble bearing surface given by this tie-plate should at least double the life of the tie.

The fixing of the rail to the plate is by steel keys in clips, offering a surface 8 inches wide instead of ¾-inch spikes or screws, and enabling a change of tie to be affected quickly when necessary without removing the rail, the fixing of the tie-plate to the tie being done beforehand, as with the chair.

The tilt of the rail to 1 in 20 can, if needed, be obtained by several means: by cutting an inclined top surface on the tie, by bending the plate for the rail flange, or by rolling the plate of varying thickness. The rail joint would be suspended, with angle fish-plates which offer sufficient bearing surface on the joint ties that the base plates would only be needed on intermediate ties.

Digitized by Google

The tie-plates now in use, about 7 inches square, are much too small for the object for which they are designed, and what is worse, require a nicety in the position of the spike holes, even to a thirty-second of an inch, to be thoroughly effective, and that is very difficult to obtain in practice.

The new plate might even be applicable to rails with flanges varying from 4 to 5 inches wide, that is rails weighing from 56 to 100 pounds per yard, different steel keys being the only material necessary for different rail sections. The plate can thus be manufactured for stock and thereby obtained cheaper, becoming a standard tie-plate.

As to the cost, compared with the chairs for the bull-head rail, it would still be somewhat less. The cost of the iron chair, weighing 40 to 50 pounds, is about $15 per ton; the base plate weighs 16 pounds and would probably cost $30 per ton, so that the plate weighs about one-third as much as the chair, and costs double the price per ton, thus representing a saving of one-third as compared with the cost of chairs.

The tie-plate would be applicable to any existing circumstances at once as the renewal of ties goes on. As it is only 16 pounds in weight as against 50 pounds for the chair, even elasticity of the road would not be impaired by laying them singly or gradually, and a good effect would be at once felt in keeping the gauge even when tie-plates were introduced on two or three ties for each rail in the track. Considered from all points, the introduction of tie-plates for safety in keeping the gauge and economy in doubling the life of the tie is to many roads of more vital importance than even increasing the weight of rail to the Goliath type, for this can be only a question for the main lines, and by many companies must be introduced very slowly indeed, while the benefit of doubling the life of the tie through application of tie-plates would be felt at once, even on branch lines.

The object sought by all railways using flange rails ought to be to come up to the perfection of the English roads using bull-head rails and cast-iron chairs, and this cannot be done without the application of steel tie-plates and an increased weight of rail to correspond with the rolling stock and speed. But to go by degrees, the tie-plate seems to be the first step necessary to adopt, and it touches not only the safety of the line but also the most important daily outlay, that is, the renewal of the ties. Besides it would save the forests and also be a medium between the adoption of steel ties and the laying of rails direct on the wooden ties.

Models of the standard tie-plate, as applied to Sandberg's 56-pound and 100-pound Goliath rail sections, using only different keys, were exhibited in the Machinery Hall (Class 61) at the Paris Exhibition.

Digitized by Google

APPENDIX B.

[Opinions on the Use of Slag Ballast.]

(R. Caffrey, Roadmaster, Lehigh Valley Railway.)—Answering your letter asking me for information as to the action of slag ballast on steel rails: I have made an examination of rails that have been in our track where slag has been used for ballast for ten years, and I find no perceptible difference in them from rails that are in our tracks near the same place and the same length of time in gravel ballast. There is no visible indication that furnace slag, such as we use for ballast, corrodes the steel rail, or has any tendency, as far as I can see or ascertain, to injure them in any way. Ballast of any kind, except, perhaps, gravel, does not generally come in contact with the rails; it is usually filled up between the ties, so that the rails bear on the ties only. This being the case, I do not see that slag or ballast of any kind can be injurious to them. It is, however, somewhat different where the base and web of steel rails are continually covered, and more or less moist; such, for instance, as in tunnels, in street and road crossings, also in yards and at water stations where engines stop to clear their fires and empty their ash pans. At these places they corrode quite rapidly, more so, I think, than iron; but outside of this I fail to see that there is any tendency to corrosion in any one kind of ballast more than in another.

(W. C. Irwin, Chief Engineer, Cleveland, Columbus, Cincinnati and Indianapolis Railway.)—In answer to your communication regarding slag ballast, would say that we have had it in use on this line of railway for six or seven years past; I have not noticed any pitting or corrosion of rails from the use of it. I am surprised, however, to find that our white oak ties do not seem to last as long in this ballast as they do in gravel by at least two years. We are now taking thousands of these ties out of the slag ballast. I am not particularly carried away by the use of this material for ballast; would prefer gravel in every case where it could be had.

(Henry C. Thompson, Chief Engineer, Cleveland and Mahoning Valley Railway.)—I agree with you that slag is about as good as broken stone for ballast, provided it is hard. In making iron for Bessemer steel an extra quantity of limestone is used to insure the removal of the silica; this produces a soft slag which possesses hydraulic properties, causing it to cement in a solid mass, or else go to dust; this is not good for ballast, but makes an excellent protection for slopes of embankments. All other "slags" are good for ballast. There may be a chemical action working detrimental to the rail, but so far as my observation goes no bad effect is visible; the rails will wear out with the traffic before any such harm is apparent. I am inclined to think it is about equal to the "ravages" of the "iron worm" said to have made its appearance in Germany some time ago.

Digitized by Google

(H. K. NICHOL, Chief Engineer, Philadelphia and Reading Railway.) —We have discovered nothing that would indicate any injurious action to the rail from the use of broken furnace slag for ballast.

(G. R. CARR, General Superintendent, Columbus, Hocking Valley and Toledo Railway.)—There has never been any appearance of any injurious effects to the rail from the use of slag ballast. We use slag from iron furnaces, some of which use charcoal, and others use bituminous coal for fuel, and limestone for flux.

APPENDIX C.

[BURNT CLAY BALLAST.]

(W. M. JACKSON, Manager, Iowa Clay Ballast Company, Kansas City, Mo.)—Any kind of clay can be used that is free from sand or grit; a small amount of sand in the clay makes it slow to burn, but it can be burned. Gumbo, or any material that is sticky and will hang together and not crumble to dust when burning, will make good ballast. Crawfish clay, low land clay, or gumbo, makes the best ballast. For burning, ground is selected where the ballast is wanted; if it is intended to burn say 25 000 yards in a pit, a fire is built in the direction of the line of track for 2 000 to 3 000 feet, as the ground will allow; a hot fire from wood and slack coal combined is started in the full length of the pit, and then covered with a layer of damp clay from 8 to 10 inches thick; then a thin coat of slack coal is spread over the clay. When the clay is heated so that the coal on the outside begins to burn, another layer of clay is put on, as at first.

The men in charge keep on digging down and building back from the line as far as required, leaving a bed of ballast from 6 to 12 feet deep, the same as a gravel pit. It is very light when burnt and easily loaded with scoop shovels. It can be called a clay charcoal or coke, and it never goes back to mud. For every ton of slack coal, from 4 to 6 cubic yards of ballast are burned, varying according to the character of the material. It absorbs moisture like a sponge, and keeps the track dry and free from dust. As to cost, where railway companies furnish the fuel, ground to burn on, side tracks at pit, and transportation for men and tools, the ballast company will burn and load for fifty cents per cubic yard. Slack coal or the refuse of the bank is used, and is the only material that will burn to good advantage. In July, 1889, the company was burning and putting the ballast under the track for the Kansas City, Fort Scott and Memphis Railway Company, at West Plains, Mo., for seventy cents per cubic yard, the railway furnishing train service; that makes twenty cents a yard for putting under the track. The company was also burning and loading for the Chicago, Rock Island and Pacific Railway for fifty cents,

Digitized by Google

the railway company furnishing all plant and putting the ballast in the track. It has also been used on the Chicago, Burlington and Quincy Railway for nine years.

APPENDIX D.

[TIES.]

The following figures will serve to show the importance of this branch of the subject. Taking the present extent of railways in the United States at 160 000 miles, and estimating an average of 2 500 ties per mile, which is a reasonably low figure, the number in many cases being 2 600 to 2 800 or even 3 000 per mile, this gives a total of 400 000 000 ties in actual use, while at an average life of eight years, 50 000 000 ties will be required annually for renewals. Averaging the annual new construction at 5 000 miles, adds 12 500 000 ties annually to the number in use, and also adds 1 562 500 ties to the number required annually for renewals. These figures are exclusive of the consumption of ties and stringers for street railways, a by no means insignificant item. Calculations of this kind are not, however, of much value, except to present clearly the great extent of the timber consumption for one item alone. The important points for each railway company are not the averages for the whole country, or the whole railway system, but are the average consumption, life, and cost upon its own lines. As an example of some individual cases, I give the following figures, taken from the annual reports of the railway companies: New York, Lake Erie and Western, 1887: 934 420 ties laid and 249 375 on the New York, Pennsylvania and Ohio. Old Colony, 1888: 315 069 new ties, 25 371 being for new second track. Michigan Central, 1888: 723 065 ties replaced. Baltimore and Ohio, 1888: 259 315 on Western division, and 609 462 on Eastern division. Illinois Central, 1888: 654 141 ties. Missouri Pacific, 1888: 1 465 121 ties. The New York Central and Hudson River Railway consumes about 800 000 ties per annum for renewals. The consumption of timber ties may be reduced either by the use of means to increase their life, or by abandoning wood for more durable material.

Increasing the life of wooden ties may be effected by the use of preservative processes, or by the use of metal tie-plates to distribute the load on the rail over a greater area of the tie than that covered by the rail flange. In this country the best results would probably be obtained by combining these two methods, as so much trouble is experienced from the narrow base or flange of the rails cutting into the wood. On some roads the ties have to be taken out of the track for this cause before they have begun to decay. The life of ties averages probably about eight years, in some cases as low as five or six, but their life in service is frequently limited by their resistance to the cutting action of

Digitized by Google

the rail flange rather than by their natural life or resistance to decay. A preservative process, therefore, which softens the wood, is of little value for ties, unless the tie-plates are used; but the most serviceable and valuable process would be one that hardens the wood as well as protects it against decay. As previously noted, some railways have already recognized the advantages of using preserved ties, and as experienced men and engineers have now turned their attention to the different processes, and have shown that successful work can be done, the use of such ties will probably become more general, especially as the increasing traffic increases the rate of the cutting of the ties. There has been until quite recently a general indifference on the part of railway men, and the extra cost is, of course, an obstacle, as railway directors, like too many people nowadays, often prefer cheapness of first cost to subsequent economy. Although extensive experiments with various preservative processes have been made in this country, and although several companies are now engaged in this business, little has been done in introducing preserved timber for regular use. Large quantities of piles and timber which are intended for use for wharves, etc., in sea water, have, however, been treated as a protection against the teredo. In railway ties comparatively little has been done; but recent improvements in methods and materials promise to result in the more extensive use of preservative processes for ties.

Creosote (either wood creosote oil or dead oil of coal tar), chloride of zinc, and sulphate of copper are the preservative mediums most generally employed; the two first named are those principally used in this country. The Chicago Tie Preserving Company, at its own works (which supply ties for the Chicago, Rock Island and Pacific Railway, under contract) and at the works erected by it at Laramie, Wyo., for the Union Pacific Railway, and at Las Vegas, N. M., for the Atchison, Topeka and Santa Fé Railway, uses the zinc process; the Southern Pacific Railway has both creosote and zinc plants at its Houston works; the Creosote Lumber Construction Company, of Fernandina, Fla., and the Carolina Oil and Creosote Company, of Wilmington, N. C., use the wood creosote oil; the Lehigh Valley Creosote Company, of Perth Amboy, N. J., Eppinger & Russell of New York, and the Old Dominion Creosoting Works, of Norfolk, Va., use dead oil of coal tar; the Louisville and Nashville Railway has a creosoting plant of its own, used for piles and timber; the Boston and Maine Railway has a kyanizing (corrosive sublimate) plant at Portsmouth, N. H., which has been used for treating hemlock ties; the Wood Vulcanizing Company, of New York, has a novel process by which the natural acids of the wood are claimed to be solidified and retained in the wood, increasing its hardness and durability. In Europe the experience with preserved ties has been very satisfactory, and such ties are almost universally used. Creosoting is the process generally employed and creosoted oak and beech ties used

Digitized by Google

on one of the railway systems in France are reported to have an average life of twenty-five years, while some are in good condition after that period.

The different effects of the different preservatives must be taken into consideration, as what may be good for timber to be used under some conditions, may be bad for that to be used under other conditions. Thus ordinary creosoting is found to soften the fibers of the wood, so that for ties in this country there would be little benefit, as the present rapid cutting by the rail flange would be increased, unless metal tie plates were used to distribute the pressure over a larger area of the tie. The zinc and wood creosote processes, however, are claimed to harden the wood. In foreign countries, where the rails are usually carried in cast-iron chairs with wide bases, this objection does not obtain, and creosoted ties are there very generally used and with excellent results. Other methods of treatment, such as the use of metallic salts, are found to harden the wood, and such methods are of course more suited for ties, while creosoting may be well adapted for timber for other purposes. In any case the operations must be carefully and thoroughly carried out, if really efficient results are to be obtained.

In regard to the use of tie-plates, these are being quite extensively adopted. In Europe they are generally large and heavy, and are often used in connection with improved fastenings. Some plates of this kind have been designed and brought out in this country, but the form of plate now generally used here is a shallow channel iron about 8 x $3\frac{3}{8}$ inches square, with the longer sides turned down, forming flanges $\frac{5}{8}$-inch deep; these flanges bite into the wood. The rail rests on the plate and the spikes are driven through holes in the plate. These plates have given good results in service and are inexpensive. A little extra expenditure in this direction effects a very material economy in maintenance and renewals.

Coming to the question of substitutes for wooden ties, I may say that for about two years I have been engaged in investigating, for the Department of Agriculture, the experience with the use of metal ties on railways in foreign countries. A preliminary report, giving the results of my investigation during the first year, was submitted in February last, and the main report, which will describe the various systems tried, is now in course of completion. Steel is the metal which gives the best results, and is most extensively used, although cast-iron bowls are still used in India and South America with sand and earth ballast. The extent to which metal ties are used is quite astonishing, and their use is continually increasing. The railway track of the future will probably consist of heavy steel rails on steel ties, but while looking for such a desirable form of track, the advantages of increasing the durability and safety of the present form of track should not be lost sight of. Metal tracks for street railways are already in use on a number

Digitized by Google

of lines in this country.—(From a paper by E. E. Russell Tratman, presented at the American Forestry Congress, Philadelphia, Pa., October 17th, 1889.)

APPENDIX E.

(Metal Track for Railways.)

In discussion of this subject, and in connection with the paper on "Maintenance Expenses of Track on Wooden and Metal Ties," by Mr. J. W. Post, Engineer of Permanent Way, Netherlands State Railways, which was read at the annual convention of 1888 (*Transactions*, June, 1888), I present the following abstract of my Preliminary Report on the "Use of Metal Track on Railways as a Substitute for Wooden Ties." This report was submitted to the United States Department of Agriculture in February, 1889, and has been published by the Department:

Europe.

England.—In England, steel ties have during the past few years been tried to a greater or less extent on quite a number of the principal lines. Foremost among them is the London and North Western Railway, which has about 50 miles of track laid with the steel cross-tie invented by Mr. F. W. Webb, the locomotive superintendent of the road. In 1888 there were 83 204 of them in use, and the experience with them had covered then six and one-half years. These ties have been experimented with on the Pennsylvania Railroad.

The general type used is the steel "inverted trough" in different forms, either rolled or stamped. As the system of track, however, includes the double-headed rail, these ties are fitted with the usual heavy cast-iron chairs to hold the rail (the Webb tie has the chairs made of steel plates), and the track is unnecessarily heavy and costly. The chief difficulty is said to be in adapting the steel tie to the double-headed rail, it being difficult to make a good piece of work. The North Eastern Railway, however, is trying steel ties under flange rails weighing 90 pounds per yard, which is a step toward the ideal track for main lines.

France.—Experiments with metal cross-ties have been made on nearly all the principal railways, and a large number of types have been tried, but several of them have been of complicated design, and therefore uneconomical. Longitudinal systems have been tried to a small extent. On the State Railways a number of trials have been made, and with some forms of tie very good results have been obtained, enabling a reduction to be made in the maintenance staff. The Paris, Lyons and Mediterranean Railway used an old type of iron tie several years ago, but abandoned it on account of the ties costing more than the wooden ties and giving a less firm and durable track; this latter defect was

Digitized by Google

probably due to the old-fashioned "gib and cotter" fastening employed. Good results have been obtained with metal ties on the Algerian system controlled by this company. The Northern Railway laid trial sections of its Belgian lines with the "Severac" and "Bernard" ties in 1885, and laid 10 000 of the former on its French lines in 1888. The Western Railway, which used the old double-headed rail, experimented in 1887 with iron ties upon which the rail chairs were cast. The Eastern Railway has tried steel ties with wooden cushions or bearing blocks under the rails, and has also tried the "Post" ties.

Holland.—In this country, probably the most extensive and most valuable, because continuous and systematic, trials have been made, and the trials have resulted in improvements which have served to develop the now well known and extensively used "Post" steel cross-tie of varying thickness, the thickness being increased under the rail seat. This tie, the invention of Mr. Post, Engineer of Permanent Way of the Netherlands State Railway Company, is economical both in construction and in maintenance, and has proved very efficient in service. The experiments have been in progress since 1865. In the early part of 1888 there were in use 457 300 ties (about 23 800 tons) of the "Post" type in Holland, Belgium, France, Germany, Switzerland, and Asia (colonies); about 272 700 ties (about 12 700 tons) more, including ties for narrow gauge railways and for the rack railway in Sumatra, were being manufactured; making a total of about 730 000 ties, or 36 500 tons.

Belgium.—On the Belgian State Railway system the "Post" tie has been laid, but it is heavier than that used on the Netherlands State Railway and heavier than the inventor considers necessary or desirable. It should be noted that it is not economical to use more metal than experience has shown to be necessary. Experiments have been made on rather a large scale. The Grand Central Railway has also had satisfactory results with metal ties. In 1873 the Superintendent of Permanent Way reported that he was fully satisfied with the experience then acquired with metal ties, and in his reports for 1886 and 1887 he stated that the favorable results had been still more marked. Metal ties of the "Coblyn" type, for light railways, have been adopted by the Société Anonyme des Chemins de Fer Économiques, and have also been tried on other lines.

Germany.—On the State railways a number of different systems of longitudinal and transverse types have been tried for several years, and some types have been regularly adopted on certain divisions. In 1887 the State railway system had a length of about 13 193 miles, with about 23 662 miles of track; of this amount about 5 530 miles had metal track—3 131 miles being laid with cross-ties, and 2 399 miles with longitudinals. Very careful records of the trials have been kept. On the Left-Bank-of-the-Rhine Railway, which comprises 1 681 miles, there are 943 miles

Digitized by Google

with metal cross-ties, and 211 miles with longitudinals, the balance being on wooden cross-ties. The first cross-ties were laid in 1876, and the first longitudinals in 1872. Since 1879 metal sleepers only have been used. On the Elberfeld division of the Prussian State railways (1 646 miles) there are 790.5 miles laid with wooden ties. 762.5 with iron ties, and 93 miles with iron longitudinals. They were laid between 1869 and 1880. The experience with iron longitudinals and cross-ties was very favorable, but still better results have been obtained since steel was introduced. Wooden ties are still used in great numbers, partly on account of their lower first cost, and partly on account of the policy of the Prussian Government to keep up the supply of timber by domestic cultivation and forest management.

Austria.—In this country longitudinals and cross-ties have been extensively used. The Northwestern Railway has a total of 59½ miles of track laid with the "Hohenegger" system, and the economy over wood is reported to be noticeable. These longitudinals have been laid in small sections year by year since 1876; the earlier ones were of iron, but the latter ones are of steel. The "Heindl" system of cross-ties is in use on a number of roads; the first were laid in 1883, and at the end of 1887 there was an aggregate of about 141 miles laid with this system of track, a considerable portion being on mountain divisions and including 6.634 miles on the Arlberg tunnel line.

Switzerland.—The Central Railway had 100 000 metal ties in use at the end of 1884, and proposed to lay 30 000 per annum till its whole system had been thus laid. The Western and Simplon Railways began using metal ties in 1883, and have been very well satisfied with them. The Gotthard Railway uses them very extensively, and they have also been adopted on the Mount Pilatus Rack Railway.

Italy.—Metal track has been used very little, if at all, and oak ties are obtainable in ample quantities and at a moderate price.

Spain.—The line from Bilbao to Las Arenas has 7.1 miles laid with steel cross-ties, and it is believed that they will prove more economical than wood. The line is 1-meter gauge. The Almanza, Valencia and Tarragona line has 251 miles laid with the De Bergue system of cast-iron plates connected by tie rods. This system is found to give greater economy, and the gauge is maintained better than with wooden sleepers. The division between Valencia and Tarragona was laid with this track in 1860, and the division between Almanza and Valencia in 1873.

Sweden.—On the State Railways about two-thirds of a mile were laid with metal ties, for experimental purposes, in June, 1886.

Denmark.—On the State Railways steel cross-ties were laid for about 18 miles in 1883–84; but the results have not been entirely satisfactory, owing to the insufficient weight and strength of the ties.

Russia.—Metal ties have only been used to a very limited extent, on two branch lines, and even there they have not been sufficiently

Digitized by Google

used to enable any reliable conclusions to be drawn from the experiments.

AFRICA.

Cape Colony.—The Cape Government Railways have some sections of the lines laid with cast-iron bowls; they are arranged in pairs and connected by transverse tie rods. The Delagoa Bay Railway, one of the new lines opening up the interior, is laid principally with steel ties.

Senegal.—About 5 000 ties of the "Severac" type have been ordered for the railways in this French colony. Cast-iron bowls are used on the French island of Réunion.

Egypt.—On the Egyptian Agricultural Railways, wrought-iron plates connected by tie rods have been used, and also cast-iron "pots" or bowls. In the English campaign of 1885, a short length of light railway of 18-inch gauge, with corrugated steel cross-ties, was laid at Suakim, but the line was soon taken up.

Algeria.—Metal ties are used on the Algerian lines controlled by the Paris, Lyons and Mediterranean Railway Company (France), and have given good results. It is estimated that the use of metal ties has saved one-fourth of the labor formerly required for maintenance.

ASIA.

India.—In this country, steel cross-ties and cast-iron bowls and plates (the latter types arranged in pairs) are very extensively used, and their use is extending very rapidly. Even in Burmah, where wood has been generally used till recently, steel ties are beginning to be introduced. There are nearly 300 miles of the State lines and a large mileage of private companies' lines laid with steel ties. The general results are reported to be good, and the ties give, on the whole, much satisfaction. They are used for lines of 1 meter and 5 feet 6 inches gauge. There are over 1,600 miles laid with cast-iron track of different types, and these also give satisfactory results in general. Such tracks have been in use for twelve or fourteen years. They give good results in reducing the work of maintenance, there being a saving of about 6½ per cent. of renewals per annum. In some of these ties wooden cushions are used for the rails to rest on.

Japan.—A few cast-iron "pot" ties were laid when the first lines were built, about 1871, but they have nearly all been taken up again and hardly any now remain in the track, while for new lines timber ties are used exclusively.

China.—Steel cross-ties are to be tried as an experiment on the new railway which was opened last year.

Digitized by Google

Australia.

Queensland.—About 40 miles have been laid with the "Phillips" type. This is a steel cross-tie intended for prairie work, where the track is laid on the surface of the ground; it is designed to be used without ballast, being simply packed with surface soil. Some years ago about 1 000 wrought-iron ties were laid, but they only lasted about five years, as they generally fractured under the rail-seats, owing, it is said, to defective fastenings; they were laid in broken stone ballast.

About 1 000 steel cross-ties were laid in 1882. In 1887, it was reported that they were in bad condition, but this may have been due to the fact that the manufacture of steel ties was in its infancy in 1882.

South Australia.—In March, 1888, metal sleepers were being laid on a new line 145 miles long.

South America.

Argentine Republic.—In this State, cast-iron "pot" sleepers are used almost exclusively, except in the far west and north. The Buenos Ayres Great Southern Railway has 13¾ miles of double track and 819¼ miles of single track laid with cast-iron sleepers of an improved design. They are adopted on account of the difficulty of procuring good hardwood ties in sufficient quantity and the greater expense of these wooden ties; also because they give a more rigid and satisfactory track. The Central Argentine Railway has 246 miles laid with cast-iron track. The Santa Fé and Cordoba Railway ordered 20 000 steel ties in England in 1888.

Chili.—Steel ties have been tried to a small extent, but the type was considered too heavy and expensive.

United States of Colombia.—There has been some talk of adopting metal ties.

Mexico.

The Mexican Railway (Vera Cruz line) is using a large number of steel ties of the type in general use in India, and has obtained very good results with them, especially at times when the road has been flooded. These ties were first used in 1884, and at the end of June, 1888, there were 46¼ miles of track laid with steel ties. The Mexican Central Railway has been contemplating the adoption of the same type of tie on the mountain division of the road, the advantages being that they last longer than wooden ties and keep the track in perfect gauge.

The "Post" Tie.

This tie is probably the most successful of all the various types of metal ties that have been put in service, and the present form of the tie is the result of many improvements, and represents several years of

Digitized by Google

experience and careful study. It is a cross-tie rolled from mild steel. (Bessemer, Thomas, or Siemens-Martin); its section is that of an inverted trough, with flaring-sides forming a section of a polygon; it is narrow and deep in the middle, the ends are closed, and the bottom edges are thickened to form a rib. One of its special features is its varying thickness, giving an ample thickness of metal at the rail seat, where the greatest strength is required, and a less thickness at the middle and ends. Thus the weight of the ties as now used is from 110 pounds to 121 pounds each, corresponding to 126.5 and 139.15 pounds if they were of uniform section. This feature represents, therefore, an economy of 15 per cent. of metal as compared with a tie with a uniform thickness equal to the maximum thickness of the "Post" tie. In the operation of rolling, the varying thickness is given and also the shape of the tie, while the bending of the ends to give the rails an inward inclination of 1 in 20 (in accordance with European practice) is done during the same operation. The narrowing of the middle portion of the tie is designed to keep the ballast from working away from under the rail seat, and in this way a stable road-bed and track are secured, thus diminishing the work of maintenance. The increased depth of this portion gives additional strength to resist bending and also offers increased resistance to creeping.

The following are the principal dimensions, given in the original metric measure and also reduced to feet and inches: Length over all, 2.55 to 2.65 meters (8.364 to 8.692 feet); width over all at rail seat, 235 millimeters (9.40 inches); width over all at middle, about 5.30 inches; width of rail seat, 110 millimeters (4 40 inches); width of end, 280 millimeters (11.20 inches); depth under rail, 74.5 to 75.5 millimeters (2.98 to 3.02 inches); depth at middle, 125 millimeters (5 inches). Thickness of cross-section at rail seat varies as follows: Thickness at bottom of flange, 6 millimeters (.24 inch); thickness at upper part of flange, 7 millimeters (.28 inch); thickness at rail seat, 9 to 10 millimeters (.36 to .40 inch); thickness at bolt holes, 12 to 13 millimeters (.48 to .52 inch); thickness at middle and ends, 6 to 7 millimeters (.24 to .28 inch). The rib on the lower edge of the flanges has a depth of about 18 millimeters (.72 inch), and projects about 13 millimeters (.52 inch) beyond the outer face of the flange. For rail fastenings reliance has been placed upon bolts, and the results have been entirely satisfactory; the fastenings keep tight, prevent vibration and rattling, and require little attention after the track has become well settled. The bolt holes are oblong, and have rounded corners. The bolt used is 91 millimeters (3.64 inches) long and 22 millimeters (.88 inch) in diameter; it has a T-head 38 by 46 millimeters (1.52 by 1.84 inches), and a cam-shaped or eccentric neck 22 by 30 millimeters (.88 by 1.20 inches), for the purpose of allowing an adjustment of gauge at curves, switches, etc. The bolt passes up through the tie and through a washer which bears on the flange of the rail and the face of

Digitized by Google

the tie; a Verona nut-lock is then put on and the nut screwed down upon it. The upper face of the washer and the lower face of the nut are indented, so as to give a good ho'd on the nut-lock. This tie presents the following advantages: Economy in material, manufacture and maintenance; efficiency in making a good track, and adjustment for gauge.

Some American Metal Ties.

The International Tie.—This is a rolled steel tie, the section of which resembles a printer's "brace" (⌐—⌐). Originally it was made in two pieces, riveted together at the middle flange, but it is now to be rolled in one piece. The dimensions are as follows: Length, 8 feet; width, 10 inches; side flanges, 2¾ inches deep; middle flange, 2 inches high; thickness, from $\frac{3}{16}$-inch at the lower part of the side flanges to $\frac{5}{16}$-inch at the middle. The middle flange is cut away in two places for the rails. The fastenings consist of flat wrought-iron clips, one on each side of the rail, which are bolted to the flange of the tie and have projections which bear upon the rail flange. Some of these ties have been in use for more than two years on the Boston and Maine Railway and the Maine Central Railway; the Long Island Railway is now giving them a trial.

The Hartford Tie.—This is a rolled steel tie, of inverted trough section, with a channel or groove along the whole length of the top table, and having the ends curved down to hold the ballast. The dimensions are as follows: Length, 7 feet 6 inches; width at top, 8 inches; width at bottom, 10½ inches; depth, 2½ inches; thickness, ⅜-inch at sides and $\frac{5}{16}$-inch at top; the channel or groove is 2½ inches wide and ⅜-inch deep. The weight is about 120 pounds. The fastening for each rail consists of two clamps ⅜-inch thick, with a hooked projection at the broad end, which holds the flange of the rail; these clamps are wedge-shaped in plan, and lie in the channel above mentioned. A bent bolt, with its head at an angle of 53 degrees with the body, is used on each side of the rail; the head is on the under side of the tie and the body passes up through the tie and clamp, the nut bearing on the inclined face of the hook of the clamp. This is the fastening as improved by Mr. Katté, M. Am. Soc. C. E., Chief Engineer of the New York Central and Hudson River Railway. By this arrangement, the bolt being at an angle, a strong grip is secured, and there is little tendency to jar the bolts loose; to prevent such loosening, however, the bolt has the Harvey grip thread, which forms a nut-lock in itself. The fastening permits of a very wide range of adjustment of gauge. These ties are being tried on the New York Central and Hudson River Railway, and careful observations will be made as to the results.

The Standard Tie.—This is a steel tie of channel section (⌊⌋) stamped to shape from a plate. The bottom is cut away at the middle, and is

Digitized by Google

bent up at an angle to offer resistance to lateral motion, the ends being open. The rail does not rest upon the vertical sides of the tie, which are cut away for a depth of three-fourths of an inch under the rail, but rests upon a block of compressed preserved wood (placed with the grain vertical). The tie is intended to be filled with ballast. The fastenings consist of two Z-shaped clips, the upper part holding the rail-flange and the lower part taking a bearing on the under side of the bottom of the tie; the upright web is nearly vertical, but slightly curved so as to grip the wood block. A bolt passes horizontally through the two clips and the block, near the top of the latter, holding all the parts firmly together. At the rail-joints it is intended to use a tie of extra width, with wide clips and two bolts, and it is claimed that this fastening will be sufficient in itself, and will obviate the necessity of using splice plates. These ties have been laid for trial on the Chicago and Western Indiana Railway. The claim is made that they are specially adapted for roads with a narrow width of ballast, owing to the resistance of lateral movement being at the middle instead of the ends of the tie.

The Taylor Tie.—This is an iron or steel tie on the "bowl" system, each tie consisting of a separate piece under each rail, connected by a third piece forming a tie-bar. The rail-bearers are short pieces of inverted trough section, placed longitudinally with the rail, and have a vertical transverse slot through which the deep flat tie-bar passes. The inside flange of the rail is held by clips, forming a part of the top table of the trough, and the outside flange is held by a hooked projection at the end of the tie-bar. No bolts or other loose parts are used.

The Toucey Tie.—This is a cast-iron "pot" tie designed by Mr. John M. Toucey, F. Am. Soc. C. E., General Superintendent of the New York Central and Hudson River Railway. Each tie consists of two "pots," of H-section, with outward flaring sides; the "pots" are connected by a tie-rod, the ends of which are bent at right angles to fit into a hole in the horizontal web, the rod passing through a hole in the side. The "pots" are 18 inches long, 9¾ inches wide on top, 16¼ inches wide at bottom, and $8\frac{5}{16}$ inches deep; the thickness varies from ¼ inch to 1 inch. The space above the web is filled with an oak block, to which the rail is secured by the Bush interlocking bolts. Some of these ties are still in use at the Grand Central Depot in New York City.

A Channel Tie.—A channel tie was used by the Pennsylvania Railway for some years subsequent to 1880. In that year some were laid on the Filbert Street extension, and in 1885 about 400 or 500 were laid on the main track in the West Philadelphia yard. The tie consisted of an ordinary 7-inch channel iron (⌐¬) 8 feet 6 inches long; the ends were closed by a piece of angle-iron riveted on, and a cross-piece of angle-iron was also riveted inside the channel, just under the outer flange of the rail. The fastenings for each rail consisted of a piece of angle-bar riveted to the face of the tie (the rivets passing through the angle-bar,

Digitized by Google

tie, and inside angle-iron) on the outside of the rail, and a loose flat clip on the inside of the rail, fastened by bolts. Writing in 1886, Mr. Brown, Chief Engineer of the Pennsylvania Railway, said: "These ties cost from $3 to $4 each. As long as we can get good oak ties for not exceeding $1 each, I would not recommend making the change, although they give perfect satisfaction and are no more trouble to keep in line and surface than wooden ties."

The Durand Tie is one of the latest patented in this country, and has been tried experimentally in France. It is similar to the Post tie, but is claimed to be lighter with equal strength, the top being corrugated lengthwise and the sides vertically. It is intended to be made of old rails rolled into sheets and stamped to shape; but, of course, ordinary steel can be used. It is 10 inches wide at the top under the rail, and 12 inches at the bottom; it is narrowed and depressed in the middle and curved at the ends. The ends may be open or closed with a movable cap. It may be $\frac{3}{16}$ inch thick, weighing 65 pounds, and costing $1; or $\frac{5}{16}$ inch thick, weighing 100 pounds, and costing $1.35. The plant for manufacturing is estimated to cost $5,000. The bolts have T heads welded to the under side of the tie, and the nuts screw down on washers of soft metal.

In Europe it is the usual practice, in addition to the use of metal track for railways, to use steel or iron longitudinals or cross-ties, or cast-iron "chairs," for street railway tracks. Steel ties are also extensively used for contractors' tracks, portable railways, narrow gauge and light railways, etc., in various parts of the world.

In connection with this subject it is interesting to note that the Railroad Commissioners of Connecticut make a reference to it in their report for 1888.

Many objections have been urged, and it has been claimed that a steel track will not be economical for the companies or pleasant for the travelers; but it can hardly be that we cannot design a successful track, when in many other countries metal tracks have been so extensively introduced, and with satisfactory results.

Digitized by Google

APPENDIX F.

[MR. FINK'S SYSTEM OF ACCOUNTS (LOUISVILLE AND NASHVILLE RAILWAY). FORM OF TABLE OF OPERATING EXPENSES.]

For what Expended.

MAINTENANCE OF RAILWAY, BRIDGES AND BUILDINGS.

1. Road repairs.—Adjustment of track.....
2. Ballast.....
3. Ditching .....
4. Culverts and cattle guards..... .....
5. Extraordinary repairs (slides, etc.).....
6. Repairs of hand and dump cars.....
7. Repairs of road tools .....
8. Road watchmen.....
9. General expenses of road department.....

10. Total roadway.....
11. Cross-ties replaced, value.....
12. Labor replacing.....
13. Train expense hauling.....

14. Total cost of cross-ties.....
15. Bridge superstructure repairs.....
16. Bridge watchmen.....
17. Shop-building repairs .....
18. Water station repairs.....
19. Section house repairs.....

20. Total cost of bridge and building repairs.....
21. General superintendence and general expenses operating department.....
22. Advertising and soliciting passengers and freight.
23. Insurance and taxes.....
24. Rent account.....

25. Total.....
26. Salaries of general officers.....
27. Insurance, taxes, and general expense.....

28. Total.....

29. Total maintenance of roadway and buildings.

STATION EXPENSES.

30. Labor, loading and unloading freight.....
31. Agents and clerks.....
32. General expense of stations, lights, fuel, etc.....

Digitized by Google

STATION EXPENSES—(*Continued*).

33. Watchmen and switchmen.....
34. Expense of switching. Engine repairs.....
Engineers' and firemen's wages.....
Expense in engine houses.....
Supervision and general expense.....
Oil and waste.....
Water supply.....
Fuel.....
35. Total.....
36. Stationery and printing.....
37. Telegraph expenses.....
38. Depot repairs.....
39. Total.....
40. Total station expenses.....

MOVEMENT EXPENSES.

41. Adjustment of track.....
42. Cost of rails, renewal value.....
43. Labor replacing rails.....
44. Train expenses, hauling.....
45. Joint fastenings.....
46. Switches.....
47. Total replacing rails.....
48. Locomotive repairs.....
49. Oil and waste used on locomotives.....
50. Watching and cleaning.....
51. Fuel used in engine houses.....
52. Supervision and general expenses in engine houses.
53. Engineers' and firemen's wages.....
54. Total engine expenses.....
55. Conductors and brakemen.....
56. Passenger car repairs.....
57. Sleeping-car repairs.....
58. Freight car repairs.....
59. Oil and waste used by cars.....
60. Labor, oiling and inspecting cars.....
61. Train expenses.....
62. Total car expenses.....
63. Fuel used by locomotives.....
64. Water supply.....
65. Total.....
66. Damage to freight and lost baggage.....
67. Damage to stock.....
68. Wrecking account.....
69. Damage to persons.....
70. Gratuity to employés.....

Digitized by Google

MOVEMENT EXPENSES—(*Continued*).

71.	Fencing burned	
72.	Law expenses	
73.	Total	
74.	Total movement expenses	
75.	Grand total for maintenance and movement expenses	
76.	Tennessee river steamers	
77.	Total expenses	
78.	Gross earnings	
79.	Net earnings	
80.	Loss	
81.	Percentage of expenses to earnings	

The figures for these tables give the total amount, and the amount per mile of road and per train mile, for passenger and freight service, for each division of the system and for the system as a whole.

Digitized by Google

INDEX TO ILLUSTRATIONS.

	Plate No.
Rails:	
Chignecto Ship Railway, 110 pounds	XXXVII.
Sayre, 110, 101, 92 and 84 pounds	"
Sandberg, 100 pounds	XXXVIII.
Philadelphia and Reading Railway, 90 pounds	XXXVII.
Pennsylvania Railway, 85 pounds	"
New York Central Railway, 80 pounds	"
Michigan Central Railway, 80 pounds	"
Dublin, Wicklow and Wexford Railway, 75 pounds	XXXVIII.
New York, New Haven and Hartford Railway, 73.66 pounds	XXXVII.
Lake Shore and Michigan Southern Railway, 71 pounds	"
Chicago and Alton Railway, 70 pounds	"
Wrenshall's bridge section	XXXVIII.
Tie Plates:	
Wrenshall's	XXXVIII.
Sandberg's	"
Cox (McConway and Torley)	XXXIX
Netherlands State Railway	"
Thomson (Pennsylvania Railway)	"
Tie Fastenings:	
Fennerty spike	XXXIX.
Davies "	"
McLean "	"
Goldie "	"
Tudor "	"
Northern Railway (France) screw	"
Netherlands State Railway "	"
Pennsylvania Railway "	"
Bush interlocking bolts	"
Fang bolts	"
Joints:	
Cox	XXXIX.
Fisher	"
Morgan	"
Long	"
Northern Railway (France)	"
New York Central and Hudson River Railway	"
Philadelphia and Reading Railway	XXXVII.
New York, New Haven and Hartford Railway	"
Chicago and Alton Railway	"
Thomson	XXXIX.
Cloud	"
Spliced-rail	"
Dublin, Wicklow and Wexford Railway	XXXVIII.
Wrenshall's, for bridge rails	"
Effect on bolts	XL.
Track Bolts:	
Northern Pacific Railway (Harvey grip)	XXXIX.
Philadelphia and Reading Railway	XXXVII.

Digitized by Google

RAIL BRACES:
Effect of wear on..... XXXVIII.

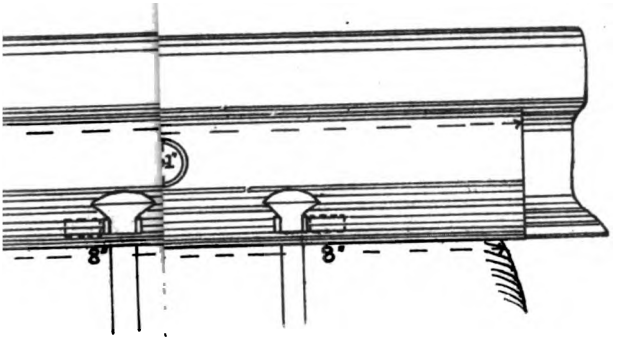
ROAD BED:
New York Central Railway..... XXXIX.
Chicago and Alton Railway ..... "

BRIDGE FLOORS:
New York Central and Hudson River Railway..... XXXIX.
English ..... "

STREET RAILWAY TRACK:
Rails and substructure ..... XLI.

Digitized by Google

CH



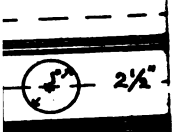


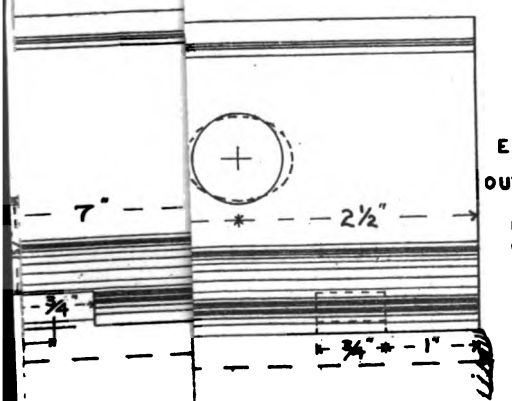
PLATE XXXVII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 433.
TRATMAN ON RAILWAY TRACK.

R.

INSIDE.

HERN

HART



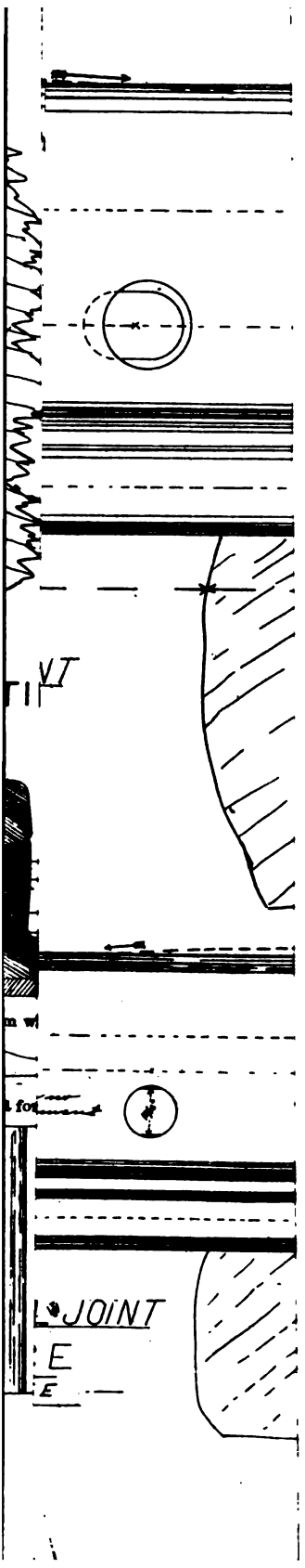
ELEVATION OF
OUTSIDE SPLICE BAR.

HOLES AND NOTCHES
OF INSIDE SPLICE BAR
ARE SHOWN DOTTED.

RAIL HOLES, 1⅛" DIA.

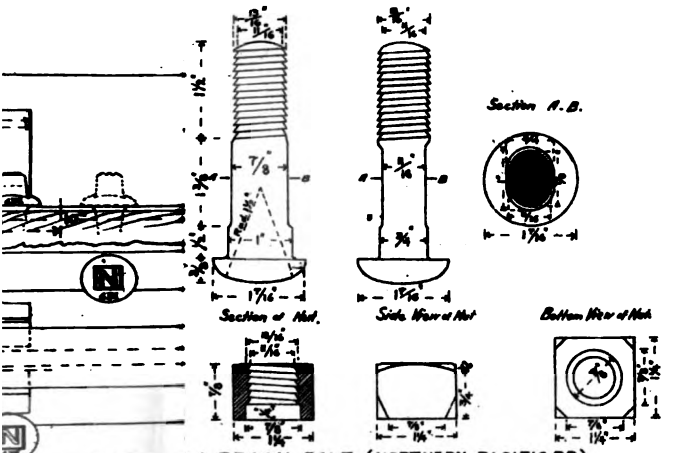
Digitized by Google

Digitized by Google



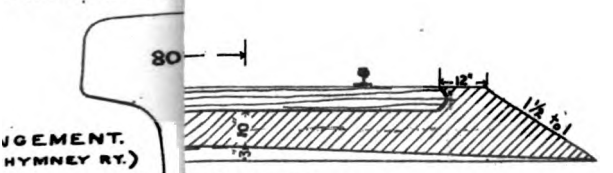
Digitized by Google

Digitized by Google

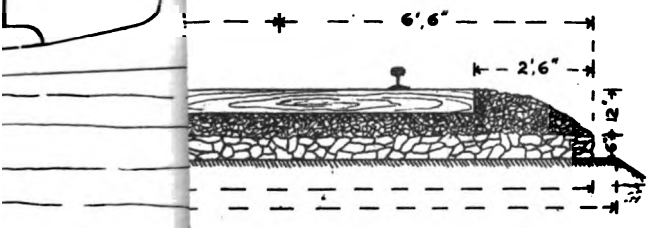


Y TRACK BOLT (NORTHERN PACIFIC RR.)

ANG-BOLT



JGEMENT.
HYMNEY RY.)



— ENGLISH IRON BRIDGE FLOORS. —

Digitized by Google

Digitized by Google

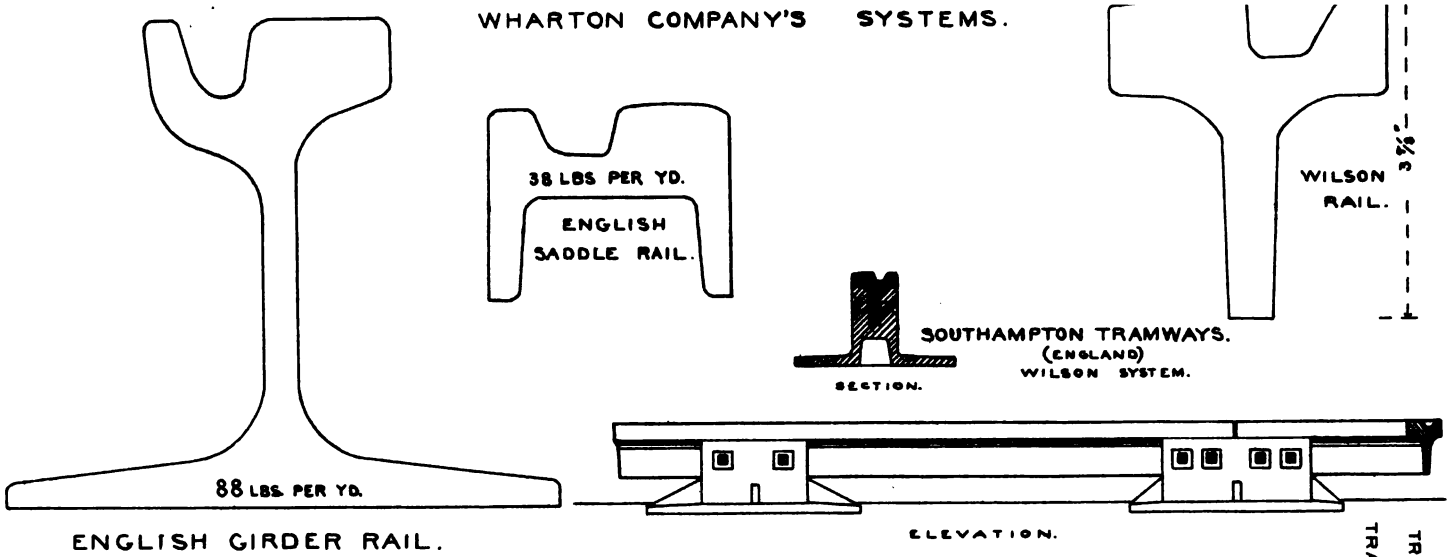


PLATE XLI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXII. Nº 433.
TRATMAN ON RAILWAY TRACK.

Digitized by Google

Digitized by Google

LOUISVILLE CEMENT.

The undersigned is General Agent for the following Standard Brands of Louisville Cement:

FALLS MILLS (J. Hulme Brand),
BLACK DIAMOND MILLS (River),
SPEED MILLS,
FALLS CITY MILLS,
QUEEN CITY MILLS,
ACORN MILLS,
BLACK DIAMOND MILLS (Railroad),
FERN LEAF MILLS,
EAGLE MILLS.

This Cement has been in general use throughout the West and South since 1830; most of the public works having been constructed with it. Orders for shipment to any part of the country, by rail or water, will receive prompt and careful attention.

Sales for 1888, 1,202,118 Barrels.
Sales for 1889, 1,338,464 Barrels.

WESTERN CEMENT ASSOCIATION,

247 W. Main St., Louisville, Ky.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

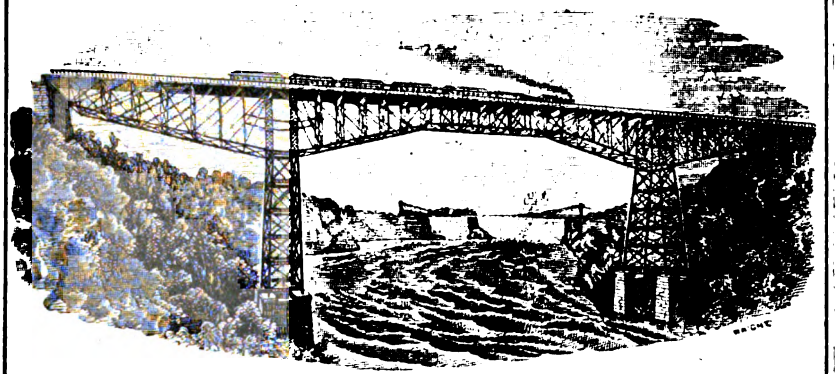
SELECT ADVERTISEMENTS WILL BE RECEIVED AT THE FOLLOWING RATES:

	TWELVE MONTHS.	SIX MONTHS.	THREE MONTHS.	ONE MONTH.
One Page.....	$100.00	$55.00	$30.00	$12.00
One-half Page.....	55.00	30.00	18.00	7.00
One-quarter Page	30.00	18.00	10.00	5.00
One-twelfth "Card".	12.00	.....	.....	....

Address the Secretary of the Society, 127 East 23d Street, New York.

Digitized by Google

UNION BRIDGE CO.



C. MACDONALD, GEO. S. FIELD,

C. S. MAURICE, EDMUND HAYES,

CIVIL ENGINEERS,

And Constructors of Iron and Steel Bridges, Viaducts, Roofs, Elevated Railroads, Marine Piers, etc.

Designs and Estimates will be sent on application to

General Office, WASHINGTON BUILDING,

1 BROADWAY, NEW YORK.

Works: { Athens, Pa. - - Capacity, 15,000 Tons.
Buffalo, N. Y. - - " 15,000 "

Digitized by Google

VIII 570

APRIL.

MAY 22 1890.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Library.

CONTENTS.

TRANSACTIONS. VOL. XXII.

No.		Page
434.	Development of the American Rail and Track. J. ELFRETH WATKINS	209
435.	Railways of Mexico. WILLIAM BARCLAY PARSONS	233
436.	The Railroad Ferry Steamer "Solano." ROBERT L. HARRIS	247

ILLUSTRATIONS.

XLII.	Fac-simile of Letter of Robert L. Stevens	209
XLIII-XLVI.	Cast-iron Rails used from 1789 to 1816	216
XLVII.	Early English Rolled Rails	218
XLVIII.	Section of Rail, Portage Railroad of Pennsylvania, 1833	218
XLIX-L.	Stevens Rail, Camden and Amboy Railroad, 1837	222
LI.	Strap Rail, Albany and Schenectady Railroad, 1837	222
LII.	Strap Rail, Baltimore and Ohio Railroad, 1837	222
LIII.	Sections of Rails	228
LIV.	Angle Splice Bar, Erie Railroad, 1857	230
LV.	Frogs and Switches used on Colliery Roads of England, 1825	232
LVI.	Ferry Steamer "Solano" in Dock	258
LVII.	Photograph of Steamer	258
LVIII.	Map showing Straits of Carquinez	258
LIX.	Cross Section and Elevation of Boat and Apron	258
LX.	Plan of Boat	258
LXI.	Plate showing Details of Construction	258
LXII.	Plan of Boat and Steering Gear	258
LXIII.	Arrangement for Burning Oil in Boilers	258

The Transactions of the Society will be sent to any subscriber at the rate of $10 per year; and to clubs of ten or more, when ordered through the Secretary of an Engineering or Technical Society or Club, who will be responsible for the payment, at 25 per cent. discount.

PUBLISHED AT THE HOUSE OF THE SOCIETY, 127 EAST TWENTY-THIRD STREET, NEW YORK, ON THE SECOND WEDNESDAY OF THE MONTH.

Entered according to Act of Congress, in the year 1890, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Digitized by Google

American Society of Civil Engineers.

OFFICERS FOR 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN, WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Standing Committees.

On FINANCE :—Charles B. Brush, Mendes Cohen, Robert Van Buren.
On LIBRARY :—Alphonse Fteley, Theodore Voorhees, William Ludlow.

Special Committees.

THE PRESIDENT OF THE SOCIETY IS HONORARY MEMBER OF ALL COMMITTEES.

ON UNIFORM STANDARD TIME :—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston, Fred. Brooks.

ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY :—F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, A. V. Abbott.

ON UNIFORM METHODS OF TESTING MATERIALS USED IN METALLIC STRUCTURES, AND ON REQUIREMENTS FOR THESE MATERIALS TO FURTHER IMPROVE THE GRADE OF SUCH STRUCTURES:—G. Bouscaren, William H. Burr, J. G. Dagron, R. W. Hunt, H. G. Morse, P. Roberts, Jr., H. B. Seaman.

MEETINGS.

Regular Meetings, on the first and third Wednesdays of the month, at 20 o'clock, except during July and August.

The rooms of the Society are open from 9 to 17 o'clock each business day, except Saturday, when they are closed at 15 o'clock.

The Library and Conversation Room will also, for the present, be open every *Wednesday Evening*, from 19½ to 22 o'clock. Members are invited to avail themselves of the opportunities afforded on these evenings, both for consultation of books and periodicals, and for conversation.

HOUSE OF THE SOCIETY—127 EAST TWENTY-THIRD STREET, NEW YORK.

Digitized by Google

At what rate will you contract to deliver at Liverpool, say from five to six hundred tons of Railway, of the best quality Iron rolled to the above pattern in twelve or sixteen feet lengths, to lap as shewn in the drawing, with one hole at each end, and the projections on the lower flange at every two feet. Cash on delivery.— How soon could you make the first delivery, and at what rate pr month until the whole is complete— Should the terms suit and the work give satisfaction a more extended order is likely to follow, as this is but about one sixth part of the quantity required. Please to address your answer (as soon as convenient) to the care of Francis B. Ogden, Consul of the United States at Liverpool

I am

Your obedient Servant

Robt L Stevens

President & Engineer of the Camden & South Amboy Rail Road & transpolation Company

nced in

AND

it cen-
ncy to
n and
ne by
f the

l dur-
locо-

l have
reater
upon

Digitized by Google

CHARLES

On FINA
On LIBE

THE P

ON U
N. Ely,

ON
MORTARS
W. W.

ON U
AND ON
OF SUCH
Hunt, H

Regu
o'clock,

The
except S
The
every W
avail th
consulta

Hou

Digitized by Google

MAY 22 18

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

434.

(Vol. XXII.—April, 1890.)

DEVELOPMENT OF THE AMERICAN RAIL AND TRACK.

By J. ELFRETH WATKINS, Assoc. Am. Soc. C. E.

In considering the vast improvements made during the present century in methods of transportation, I think there has been a tendency to over-estimate the benefits which have arisen from the invention and improvement of the locomotive, and to overlook what was done by those who devoted time and thought to the development of the American system of permanent way.

The slight improvement made in track construction in England during the first quarter of the century, made the introduction of the locomotive there possible.

Trevithick's locomotive of 1804, crude although it was, would have been much more successful and would have brought him much greater fame, as one of the first inventors of the locomotive, had the track upon which it ran been constructed according to modern methods.

Digitized by Google

In this matter, as in many others, history has repeated itself. As the improved roads in the past made it practicable to transfer the burden from the pack mule to the wheel vehicle, and the traveler from the saddle horse to the light, comfortable and rapidly moving carriage, so the development of the iron railway of the nineteenth century has made it possible for us to enjoy the safety, speed, and comfort of the express train of to-day, drawn by the fleet and powerful locomotive.

Even before the locomotive was a practical machine the advantages of the cast-iron tram-road were fully appreciated.

By careful calculation a distinguished London engineer, in 1802, found that while it cost 3s. 4d. per ton, per mile, to transport bulky freight over turnpikes, the cost on iron horse tram-roads was 4d., just one tenth.

George Stephenson, while President of the "British Carrying Companies," stated "that by the introduction of the horse tram-road the monthly expense of that company for coal carriage alone, had been reduced from £1 200 to £300.

An edition of "Wood's Treatise on Railroads," published in 1830, which was one of the earliest and most reliable standard works on railroad subjects, calls attention to the economical operation of the coal railroad, 9 miles long, near Mauch Chunk, Penn., then operated by horse-power, and states that by this method "it has repaid its whole cost since 1827."

And the modern street railway, operated by horse-power, is able to earn handsome dividends by carrying passengers, often for a greater distance, at a lower fare than the steam cars.

Before taking up the question of the development of rails and track, let us look into the circumstances that led to the construction of the early iron railways, and particularly to the relations which existed between coal and iron, and the railroad in the beginning.

Coal, Iron and the Railroad.

That "necessity is the mother of invention," is nowhere better exemplified than in the circumstances that led to the birth and growth of the railroad.

The demand for that great necessity, coal—a necessity that became more and more urgent as the forest disappeared to satisfy the demands

Digitized by Google

of a dense population—first directed thought toward devising improved methods for transporting it from the collieries of Great Britain to the adjacent navigable streams or near seaports.

Coal had been mined in England as early as the middle of the ninth century.

Henry III, in 1259, granted the privilege of digging coal to certain persons in Newcastle, and by the beginning of the fourteenth century, coal had become an article of export. At that time it was called "sea coal," owing to the fact that it was shipped by vessel to various ports.

Early Use of Iron.

There can be no doubt that good methods of making iron were known to the ancients.

The Bible bears evidence in many texts to the high esteem in which the iron worker was held. Tubal Cain, supposed to be of the seventh generation from Adam, is described in Genesis iv, as "an instructor of every artificer in brass and iron;" and again we find, in speaking of the Israelites in Deuteronomy iv, the statement: "For the Lord hath taken you and brought you forth out of the iron furnace, even out of Egypt."

Processes of making iron were known to the Babylonians and the Assyrians. The stones in the celebrated bridge said to have been built by Nitocris, the daughter of Nebuchadnezzar, were held together by bands of iron, kept in place by molten lead.

The Phœnicians, Persians, and even the Chinese were acquainted with processes of forging iron centuries before the Christian Era; and in India, in the Temple of Cuttub at Delhi, there stands a pillar of solid forged iron over 16 inches in diameter and nearly 60 feet high, supposed to have been erected in the third century.

But these methods must be included among the lost arts—arts lost in the great abyss of the middle ages, which swallowed up so many of the results of the skill and ingenuity of the ancient world.

Both among the Greeks and the ancient nations of the Orient, as we learn from Homer, the early historians and the latest inscriptions and archæological discoveries, iron was once regarded as a precious metal. Homer's elaborate description of the shield of Achilles, forged by Vulcan, undoubtedly shows that the art of working iron was fully understood in that semi-fabulous epoch.

Digitized by Google

It is probable that iron first came into common use in the arts and manufactures when Spain flourished under the Visigoths, who derived it from their ancestors, the Scythians. Spanish iron brought high prices for many years.

The Iron Industry in England.

Early in the fifteenth century, many blast furnaces were in existence in France, and soon afterward they were introduced in Sussex, Kent and Surrey, in England. Swank, in his "History of Iron in all Ages" (page 34), tells us that "this event gave a fresh impetus to the iron industry" of England.

As the processes of extracting iron from various ores became more fully understood, the demand increased; and in order to keep up the supply great inroads were made each year upon the forests for fuel.

During the reign of Queen Elizabeth (1558–1603), the iron industry had assumed such proportion, that the consumption of wood became a very serious matter, iron being then smelted exclusively by charcoal.

The destruction of the forests had been so rapid that Acts of Parliament were passed in 1558, 1581, 1584, restricting the cutting of wood for charcoal, and the iron industry languished for over a century.

In the meantime thought had been directed to the processes of smelting iron with pit coal. Sturdevant's method, although patented in 1611, was not practicable; and Dudley, who eight years afterward solved the problem with some success, was so much abused by the charcoal smelters, that fearing bodily injury, he too abandoned the business. Nothing further seems to have been done towards using coal for smelting iron ore in England during the seventeenth century.

The Iron Industry in America.

As early as 1621, a considerable quantity of iron was produced in Virginia, that colony leading the industry until 1628, when Massachusetts forged ahead.

Wood fuel being plenty in America, the industry grew rapidly, so rapidly that Parliament passed an Act in 1660, prohibiting the use of any but English ships in the exportation of iron from the colonies; and in 1679, a duty of 10s. per ton on pig iron was imposed by the British Government.

In 1750, about 3 500 tons of pig iron having been imported into Eng-

Digitized by Google

land from America, a law was passed by Parliament removing this duty, but prohibiting all persons in the colonies, under penalty of £200, from erecting a forge, or working a tilt hammer or a rolling mill. This was one of the "grievances" that instigated the Declaration of Independence.

The eminent historian Bancroft, commenting on this fact, says: "America abounded in iron ore; its unwrought-iron was excluded from the English market, and its people were rapidly gaining skill at the furnace and the forge. In February, 1750, the subject engaged the attention of the House of Commons.* After a few days' deliberation a bill was brought in which permitted American iron in its rudest forms to be imported duty free; but now that the nailers in the colonies could afford spikes and large nails cheaper than the English, it forbade the smiths of America to erect any mills for slitting or rolling iron, or any plating forge to work with a tilt."

In 1761, less than 17 000 tons of iron had been made in all Great Britain, and over 4 500 tons had been imported from America.

Coal Mine Tram-Roads.

The earliest railways in England were laid in the coal mines, and from the mines to the adjacent water courses. These ways consisted of squared timber rails laid in the ground, held to gauge by cross timbers, to which they were fastened by wooden pins.

Roger North, in 1672, in his biography of his brother Francis, the Lord Chancellor, describes a wooden railway which he had seen at Newcastle, during the reign of Charles II, as follows: "The manner of the carriage is by laying rails of timber from the colliery down to the river exactly straight and parallel, and bulky carts are made with rowlets fitting these rails, whereby the carriage is so easy that one horse will draw 4 or 5 chaldrons of coals." The Newcastle chaldron weighed 5 936 pounds, so that one horse hauled 8 or 9 tons.

*The exact wording of the act as finally passed was as follows: "And that pig and bar iron made in His Majesty's colonies in America may be further manufactured in this kingdom, be it further enacted by the authority aforesaid, that from and after the 24th day of June, 1750, no mill or other engine for slitting or rolling of iron, or any plating forge to work with a tilt hammer, or any furnace for making steel, shall be erected, or after such erection continued in any of His Majesty's colonies in America; and if any person or persons shall erect, or cause to be erected, or after such erection continue, or cause to be continued, in any of the said colonies, any such mill, engine, forge, or furnace, every person or persons so offending shall, for every such mill, engine, forge, or furnace, forfeit the sum of £200 of lawful money of Great Britain."

Digitized by Google

Coal was mined in America as early as 1770, on the James River in Virginia; and was used at the Westham foundry to manufacture shot and shell during the Revolutionary War.

In 1825 coal mining had commenced to be an industry in the Schuylkill and Lehigh regions. In this country, as in England, the earliest railroads were built in and from the coal mines, first at Mauch Chunk, Honesdale, and Pottsville in Pennsylvania, and Chesterfield in Virginia, to the nearest navigable streams.

The first locomotive that ever turned a driving wheel on a railroad on the Western Continent was imported from England in 1829, for use on the Delaware and Hudson Canal Company's road at Honesdale, Penn. I refer to the locomotive "Stourbridge Lion," a full size model of which can be seen in the Section of Transportation and Engineering in the United States National Museum.

As the supply of coal was increased by improved methods of mining and cheaper means of transportation, it gradually superseded charcoal in the manufacture of iron. The cost of pig iron was reduced from £16 10s. in 1660 to £3 in 1760, and the price did not vary much from this until the American Revolution cut off the supply of iron that England had been receiving from the colonies. This was several years previous to the introduction of steam pumping engines, which between 1775 and 1790—through the improvements and inventions made by Watt, in the engines of Savery and Newcomen—reached such a degree of perfection that good steam pumps were put in every prominent colliery, so that the amount of coal mined reached enormous proportions as the cost of mining it was lessened.

Iron Furnaces in England.

The following tabular statement shows the growth of the iron industry in England during 85 years prior to the introduction of the locomotive, in 1825:

		Number of Furnaces.	Production, Tons.
1740	Charcoal iron.....	59	17 350
1788	Charcoal iron.....	24	13 100
	Pit coal (coke).....	53	48 800
1796	Pit coal (coke).....	121	124 879
	Charcoal.....	none.	
1802	Pit coal (coke).....	168	170 000
1806	Pit coal (coke).....	227	250 000
1825	Pit coal (coke).....	305	600 000

Digitized by Google

Thus coal, iron and the railroad, even before the successful introduction of the locomotive, had become three equally important pillars in the foundation of the great systems of transportation, upon which has been built the splendid structure of national development and commercial prosperity that has made the higher civilization of to-day possible.

Cast-Iron First Used for Rails in England.

The price of iron was materially reduced as coal became cheap and plenty, and it at last became economical to use it in the construction of rails. It is probable that the earliest iron used in track construction was cast in plates 3 or 4 feet long, 2 or 3 inches wide and ½ or ¾ of an inch thick. These plates were doubtless spiked on top of the wooden stringer rail where the wear was the greatest.

But timber was dear in England at the close of the last century, and many attempts were made to devise a cast-iron rail that should be suited to the traffic of the English tram-roads.

We have several models* of the cast-iron rails which were used from 1789 to 1816. From them a fair impression can be obtained of the crude ideas that the early English tram-way contractors had in regard to rails.

Fig. 1. *Cast-iron Edge Rail*, 1789. Patented in England by William Jessop, engineer. Laid on a road in Loughborough. The rail was fish-bellied, and at first was not supported by a chair, the wood or stone block being hewn to fit the end of the rail. Near the ends the rails had a flat projecting base, in which there were holes for the bolts which fastened them to the wooden block or sleeper. (Plate XLIII.)

Fig. 2. *Cast Edge Rails*, 1797, with joints supported by chairs (first chairs adopted). These iron chairs were cast the reverse of the ends of the rail, two bolts through the stem of the rail at each joint. Adopted on the Lawson Main Colliery Road, Newcastle-on-Tyne, England, by Mr. Barnes; first supported by wooden but finally by stone blocks. (Plate XLIII.)

Fig. 3. *Cast Edge Rails*, 1802. Four feet 6 inches long. Invented by Mr. Wyatt. Used on the railway at the slate quarry, Lord Penrhyn's estate, near Bangor, North Wales. The general shape of the cross-section of this rail was a hexagon. At each end of the rail a dove-tail block, 2 inches long, was cast on the bottom. This was slipped into a chair which had previously been attached by a bolt to the wooden or stone support. (Never came into general use.) (Plate XLIII.)

* Drawings from these models are given below. See Plates XLIII-XLVI.

Digitized by Google

Fig. 4. *Cast Tram Rail*, 1803, "with flange higher in the middle and a nib under the tread to add strength." Used on the Surrey Railroad, England. These rails had a rectangular notch (half square) in the ends, the joints being completed by one square-headed iron spike (countersunk). (Plate XLIV.)

Fig. 5. *Cast Rail with Concave Top*, 1803. To be used by general road wagon and to be embedded in common roads. Patented by Josiah Woodhouse. This rail was fastened to transverse cross ties by bolts slipped into slits through the base. (Plate XLIV.)

Fig. 6. *Cas Tram Rail*, 1804, 3 feet long. From the track from Penydarren Works to Glamorgan Canal, near Aberdare Junction, Wales, upon which Trevithick's first locomotive, to help man, ran in 1804. This original rail was a gift of J. W. Widdowson, Esq., London, Eng., to the United States National Museum, where it is on exhibition. (Plate XLV.)

Fig. 7. *Cast Tram Rail*, 1808, laid without bolts or spikes. Charles Le Cann, of Llannelly, Wales, received a premium of twenty guineas from the Society of Arts for the invention of this rail, which was ingenious in construction. Projecting pins, pyramidal in shape, were cast on the bottom of this tram-rail at the points where the stone supports came under the rail; the joints being dove-tailed into each other, dispensed with the need of any other form of joint fixture. These rails were about 5 inches wide, and weighed 42 pounds per yard. (Plate XLVI.)

Fig. 8. *Cast Rail*, 1816; patented by Losh & (George) Stephenson, of Killingworth, Eng. With half-lap joint, through which a horizontal pin passed transversely, joining the rails together, and at the same time fastening them to the cast-iron chair. A large portion of the Stockton and Darlington Railroad was laid with this rail. (Plate XLVI.)

Frontispiece: Rail Rolled with Convex Top, Stem, and Base, 1830 (generally used on American railroads since that time; original section). This rail was fastened to stone blocks with hook-headed spikes; at the joints were iron tongues fastened to the stem of the rail by rivets put on hot. Rails rolled in lengths of 18 feet, weighing 36 pounds to the yard. The first standard of the Camden and Amboy Railroad, and first rail ever rolled with a base. Designed by Robert L. Stevens, president Camden and Amboy Railroad; manufactured by Guest & Co., Wales, in 1830. (Plate LXII.)

Digitized by Google

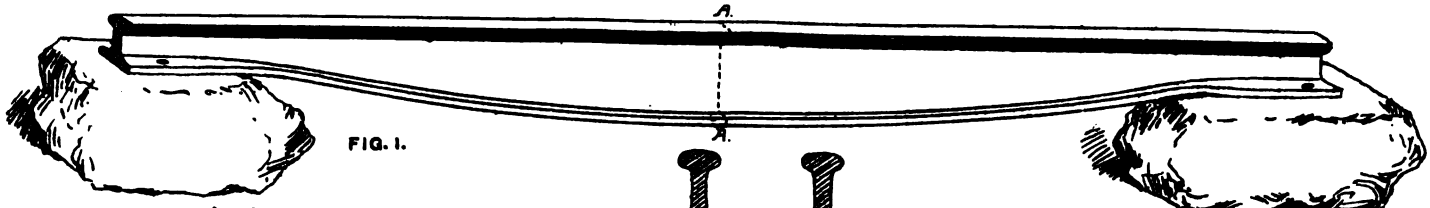
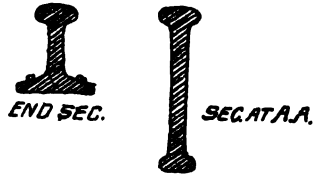


FIG. 1.

WM. JESSOP'S
PATENT RAIL
1789.



END SEC. SEC. AT A.A.

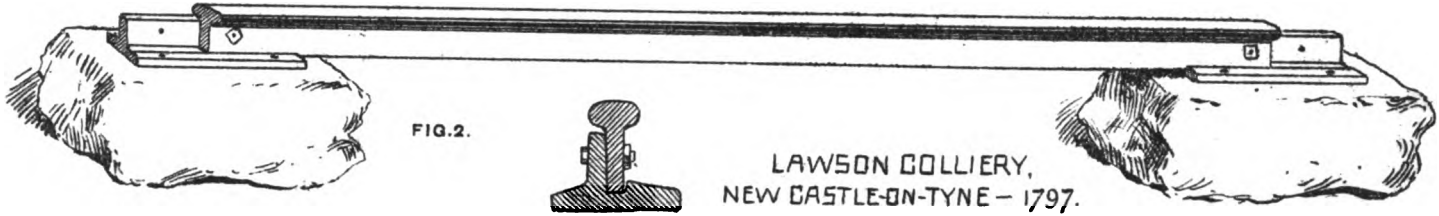


FIG. 2.

SEC.

LAWSON COLLIERY,
NEW CASTLE-ON-TYNE – 1797.

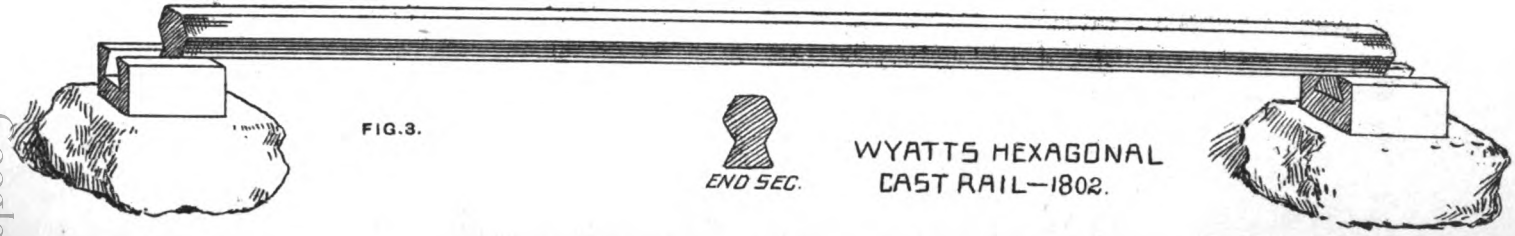


FIG. 3.

END SEC.

WYATTS HEXAGONAL
CAST RAIL—1802.

PLATE XLIII.

VOL. XXII. P. 216

Digitized by Google

Digitized by Google

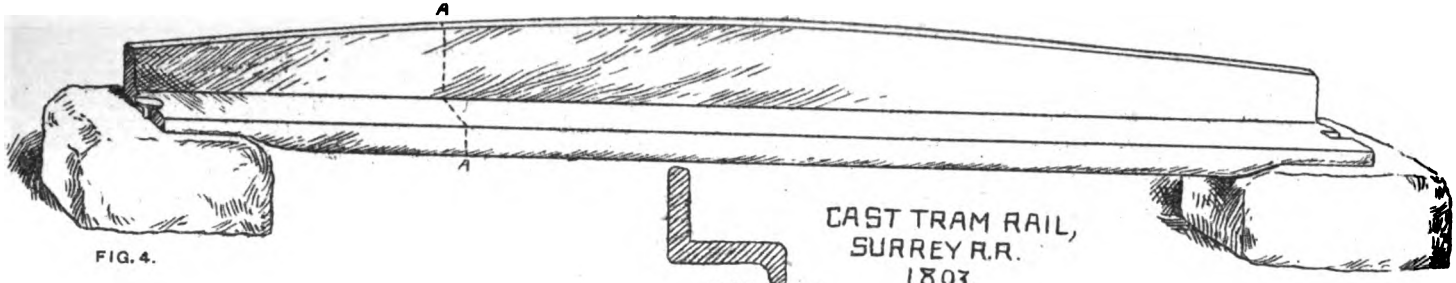
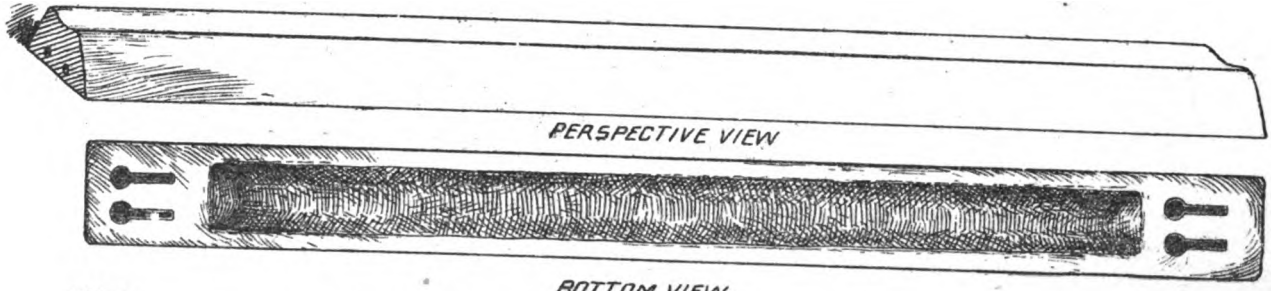


FIG. 4.

CAST TRAM RAIL,
SURREY R.R.
1803.

SEC. AT A.A.



PERSPECTIVE VIEW

BOTTOM VIEW

FIG. 5.

WOODHOUSE'S CAST RAIL-1803.

PLATE XLIV.

VOL. XXII, p. 265.

Digitized by Google

Digitized by Google

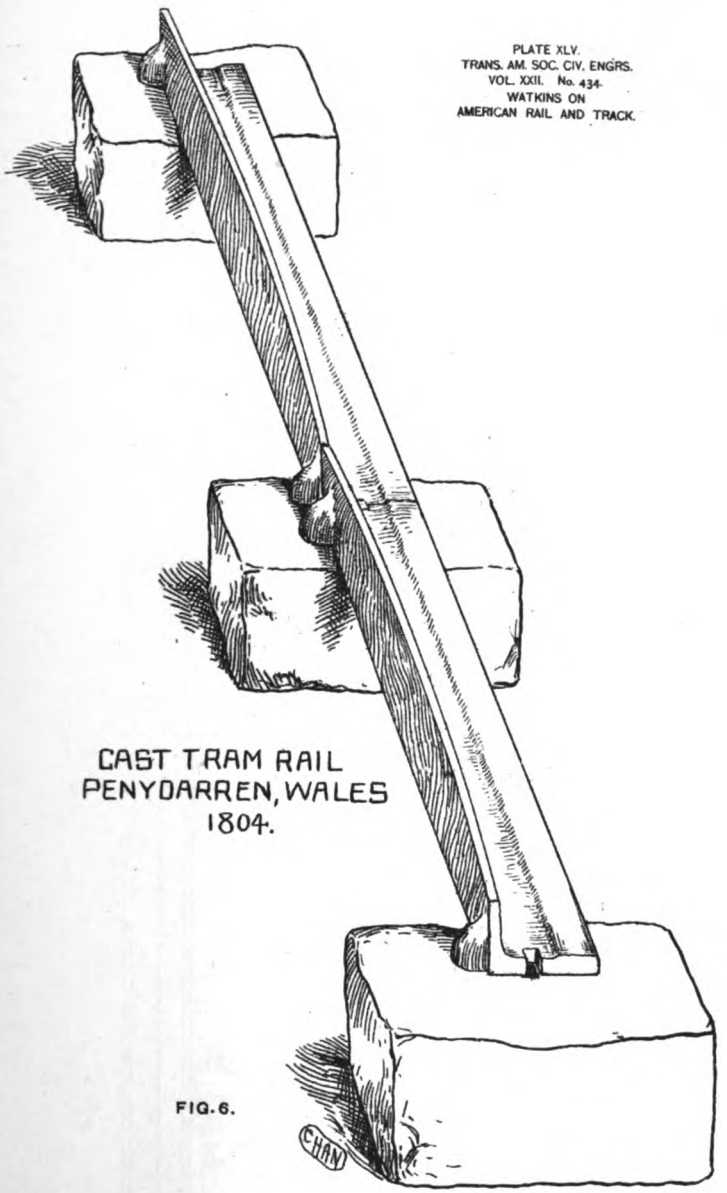


FIG. 6.

Digitized by Google

Digitized by Google

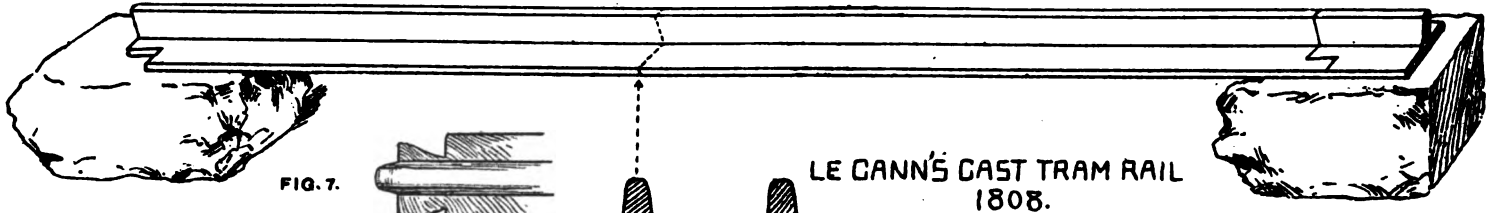


FIG. 7.

TOP VIEW. SECTION. SECT. THRO' SPIKE.

LE CANN'S CAST TRAM RAIL
1808.
(DOVE-TAIL JOINT.)

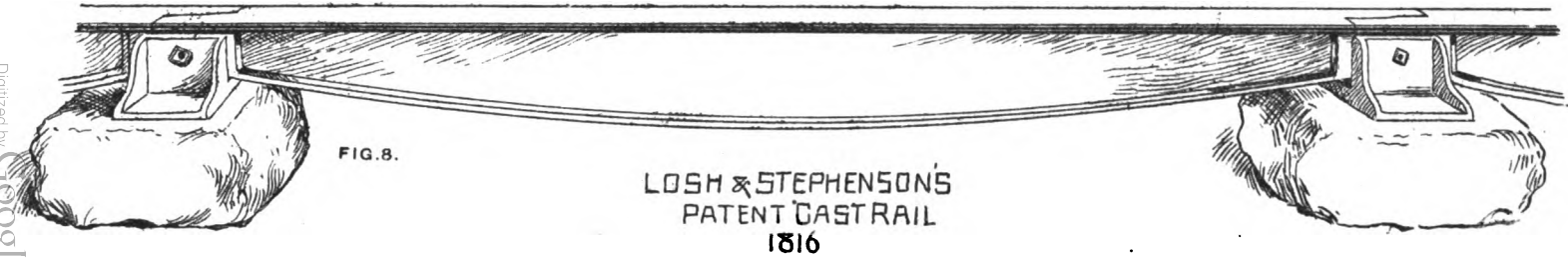


FIG. 8.

LOSH & STEPHENSON'S
PATENT CAST RAIL
1816

Digitized by Google

Digitized by Google

Rolled Iron Rails Introduced.

Early in this century inventive genius increased the power of the stationary engine and the efficiency of the steam blast and of the machinery for working and handling iron.

The puddling furnace, first used in 1784, was rapidly improved by Henry Cort about the beginning of the century. He also invented and introduced the rolling mill about the same time, so that it became possible to roll iron rails cheaply. These were at first rolled in lengths of about 12 feet. We have models of these early English rolled rails. Drawings from these are as follows:

Fig. 9. *Wrought-iron Rail*, 1811, with rectangular section, about 2 x 1¼ inches, used at Lord Carlisle's Coal Works at Tindell Fall, Cumberland, Eng. (Plate XLVII.)

Fig. 10. *Wrought-iron Rail*, 1820; patented by John Birkenshaw, of the Bedlington Iron Works, England. A clause in the patent specifications reads: "The upper surface to be slightly convex to reduce friction. The upper part to rest on supporting blocks, chairs and sleepers. The wedge form is used because the strength of a rail is always proportioned to the square of its breadth and depth. Hence this (wedge) form of rail possesses all the strength of a cube equal to its square. The joints are made with a pin." Birkenshaw's invention was ingenious in designing the rolls by which these rails were fairly rolled in lengths of 18 feet. Cast bars were soon after dispensed with. (Model is made from drawings and specifications, English patent No. 4503, to John Birkenshaw, sealed October 23d, 1820.) (Plate XLVII.)

Fig. 11. *Wrought-iron Edge Rail*, 1829, with fish-bellied web. These rails were used by Stephenson in laying the Liverpool and Manchester Railway. Chairs used at joints; rails 15 feet long, supports 3 feet apart, weighed 35 pounds per yard. (Plate XLVII.)

Fig. 12. *Rail* (original), laid Portage Railroad, Pennsylvania. Imported from England 1832; laid 1833. It was the original design to lay the whole Portage Railroad with stone blocks and T-rail. (Plate XLVIII.)

Digitized by Google

The Beginnings and Development of American Track.

In 1825–27 a few isolated coal tram-roads existed in the mining regions in Pennsylvania and Virginia, and in the stone quarries in Massachusetts. These roads were laid with wooden rails, capped with thin merchant bar iron. It was about this time that the Pennsylvania Society for the Promotion of Internal Improvement sent an engineer abroad to examine the English railways. The report made by William Strickland, published (fully illustrated) during the year 1826, shows that rapid advances in track construction had been made in Great Britain during the preceding decade, notwithstanding the fact that comparatively few locomotives were at work and no steam railway for general traffic had been opened.

This report, without doubt, contained the most trustworthy information obtainable at that time by American railway projectors.

But America presented a very different problem from England to the pioneer railway builders. England was an old country, rich in commerce and foremost in manufacture, of comparatively small area and very densely settled, having a population of nearly two hundred to the square mile of territory. At that time the population of the whole United States was less than four to the square mile. The seven States, Connecticut, Massachusetts, New York, New Jersey, Pennsylvania, Delaware and Maryland, where most of the early railroads were projected, had an average population of a little over thirty-five to the square mile.

English and American Engineers.

The British railway projectors had the advantage of being able to call into their service a trained force of civil engineers. These engineers were generally connected with well organized scientific societies, and were experienced in the construction of public works, and familiar with what had been done for years on the coal tram-roads: men on whose judgment the wealthy capitalist was willing to stake the money for the proposed improvement. England had numerous machine shops, fairly well equipped with tools and stationery engines, and many coal mines and iron foundries in operation. It was thus possible then to obtain without difficulty the material for laying the tracks with heavy rails, firmly attached by strong chairs to the sleepers that were imbedded in stone ballast.

Digitized by Google

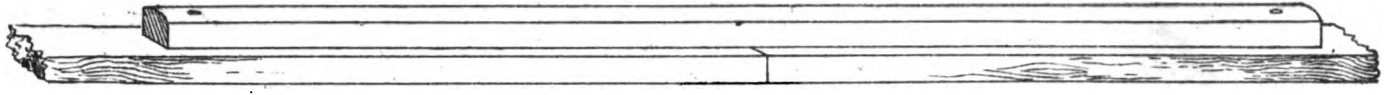


FIG. 9.

LORD CARLISLE'S WROT IRON RAIL
1811.

FIG. 10.

WROUGHT IRON RAIL
PAT'D BY JOHN BIRKENSHAW.
1820.

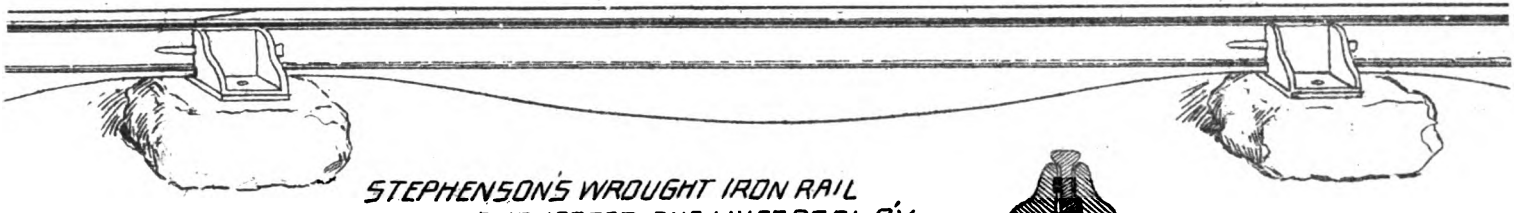


FIG. 11.

STEPHENSON'S WROUGHT IRON RAIL
LAID ON MANCHESTER AND LIVERPOOL R'Y.
1829.

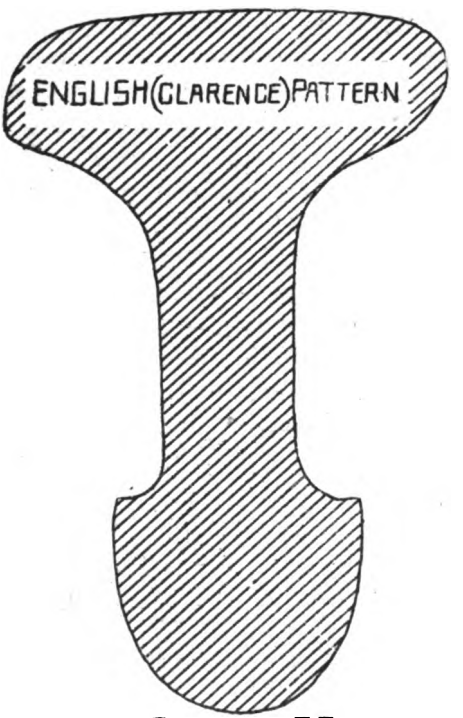
SEC. THRO' CHAIR.

PLATE XLVII.

VOL. XXII. p. 28.

Digitized by Google

Digitized by Google



PORTAGE R.R.
OF PENNSYLVANIA 1833.

FIG. 12.

Digitized by Google

Digitized by Google

With the exception of making the rail heavier, and using steel instead of iron, and substituting an iron for the wooden cross-tie and improving the splice chair, there has been no great change in the English system of track laying in the last fifty years.

The civil engineers who were first called into the service of the American railroads were generally men connected with the army, who had obtained their education at West Point, the only institution in the United States where engineering was taught at that time, or they were canal or road engineers who had obtained a little experience in canal and turnpike construction. On the railroads that they built the curves and gradients were frequently sharp and steep, as few cuts or fills were made. These cheap roads were quickly extended, through a rapidly growing country, with a view to connect the navigable water courses, and to unite with the steamboat companies in forming systems of intercommunication. By the aid of these roads the Western and Southern States rapidly increased in population and commercial prosperity. In 1832 the South Carolina Railroad from Charleston to Hamburg, which was then the longest railroad in the world, 135 miles long, was one long trestle work, with rails of squared timber, capped with strap iron, framed to the tops of posts, where grading would have been necessary.

America without Rolling Mills at the Beginning of the Railroad Era.

When the corner-stone of the Baltimore and Ohio Railroad was laid in 1828 there was not a rolling mill in all the United States where rails of the character laid on the Stockton and Darlington railroad could be rolled; in fact, the only rails rolled in America for several years after was the strap rail of merchantable bar iron 2½ inches wide and $\frac{5}{8}$ of an inch thick, the holes for the spikes often being drilled by hand. The roads, such as the Camden and Amboy, in New Jersey, Boston and Providence, Philadelphia and Germantown, and the Pennsylvania (then under State control), which did not adopt this construction, were compelled to obtain their edge rails and rail fastenings from England.

The half century from 1825 to 1875 may be called the experimental era of the American railroad, since the experience obtained during that time has finally led to the adoption throughout the whole country of an almost uniform standard of track construction (depending of course

Digitized by Google

upon the traffic). To the changes in form and the development of the American rail during this period I particularly invite your attention.

The First Rail Rolled With a Base.

From an examination of the minutes of the Board of Directors of the Camden and Amboy Railroad, September, 1830, I find that in the instructions given to Robert L. Stevens, President and Chief Engineer of that Company, who had been ordered to visit England to inspect and report upon railroad matters there, he was directed to purchase "all iron rail," which the management of that company preferred to the wooden rail plated with strap iron.

Mr. Stevens sailed a few days later, and it was during this voyage that he designed the first rail ever rolled with a base, whittling several cross-sections out of wood, which he obtained from the ship's carpenter.

He was familiar with the Birkenshaw rail, with which the best English roads were then being laid, but he saw that, as it required an expensive chair to hold it in place, it was not adapted to our country, where metal workers were scarce and iron was dear. He added the base to the T-rail, dispensing with the chair. He also designed the "hook-headed" spike (which is substantially the railroad spike of to-day) and the "iron tongue" (which has been developed into the fish-bar), and the rivets (which have been replaced by the bolt and nut) to complete the joint.

I have here a fac-simile * of the letter which he addressed to the English iron masters a short time after his arrival in London. The letter contains a cross-section, side elevation and ground plan of the rail, for which he requested bids. His letter reads:

"Liverpool, November 26th, 1830.

"Gentlemen,—At what rate will you contract to deliver at Liverpool, say from 500 to 600 tons of railway, of the best quality iron rolled to the above pattern in 12 or 16 feet lengths, to lap as shown in the drawing, with one hole at each end, and the projections on the lower flange at every 2 feet, cash on delivery?

"How soon could you make the first delivery, and at what rate per month until the whole is complete? Should the terms suit and the work give satisfaction a more extended order is likely to follow, as this is but about one-sixth part of the quantity required. Please to address your

* See Plate XLII.

Digitized by Google

answer (as soon as convenient) to the care of Francis B. Ogden, Consul of the United States at Liverpool.

I am,

Your obedient servant,

ROBERT L. STEVENS,
President and Engineer of the Camden and South Amboy Railroad and Transportation Company."

The base of the rail which he first proposed was to be wider where it was to be attached to supports than in the intervening spaces. This was afterwards modified, so that the base was made one width, (3 inches), throughout. Mr. Stevens received no favorable answer to his proposals, but being acquainted with Mr. Guest (afterwards Sir John Guest), then a member of Parliament and proprietor of large iron works in Dowlais, Wales, he prevailed upon him to have the rails rolled at his works. Mr. Guest became interested in the scheme and accompanied Mr. Stevens to Wales, where the latter gave his personal supervision to the construction of the rolls. After the rolls were completed the Messrs. Guest hesitated to have them used, through fear of damage to the mill machinery, upon hearing which Mr. Stevens deposited a handsome sum guaranteeing the expense of repairing the mill in case it was damaged. The receipt for this deposit was preserved for many years among the archives of the Camden and Amboy Company. As a matter of fact, the rolling apparatus did break down several times. "At first," as Mr. Stevens in a letter to his father, which I have seen, described it, "the rails came from the rolls twisted and as crooked as snakes," and he was greatly discouraged. At last the mill men acquired the art of straightening the rail while it cooled. The first shipment, consisting of 550 bars, 18 feet long, 36 pounds to the yard, arrived in Philadelphia on the ship Charlemagne May 16th, 1831. The weight of the next shipment, several months afterwards, was increased to 42 pounds per yard, the rail being 3½ inches high. Over 30 miles of this rail was immediately laid down.

A few years after,* on much of the Stevens rail laid on the Camden and Amboy Railroad, the rivets at the joints were discarded and the bolt with the screw thread and nut, similar to that now used, was adopted as the standard. (See Fig. 13, Plate XLIX.)

Fig. 14 (Plate L) shows how this rail was used on a superstructure on the piling through meadows and marshy ground.

* See Stevens' "Engineering in America," 1837.

Digitized by Google

The Stevens rail did not come into general use for several years, the next road to adopt it being the Boston and Providence, about 1840.

On the Boston and Lowell Railroad, Massachusetts, the fish-bellied rail was laid in chairs on stone blocks.

The Albany and Schenectady road consisted of strap rail laid on longitudinal sleepers of wood, supported on trenches filled with broken stone. (See Fig. 15, Plate LI.)

On the Baltimore and Ohio Railroad, and the Columbia road in Pennsylvania, the strap rail was attached to the edges of stone blocks, which were laid on trenches filled with broken stones; the corners of the stone stringers were chamfered. (See Fig. 16, Plate LII.)

On the Pennsylvania Portage Road, the English (Clarence) rail was laid in chairs (1833). As late as 1847, the Hudson River Road used the Stevens rail, supported by chairs, but these were soon afterwards discarded.

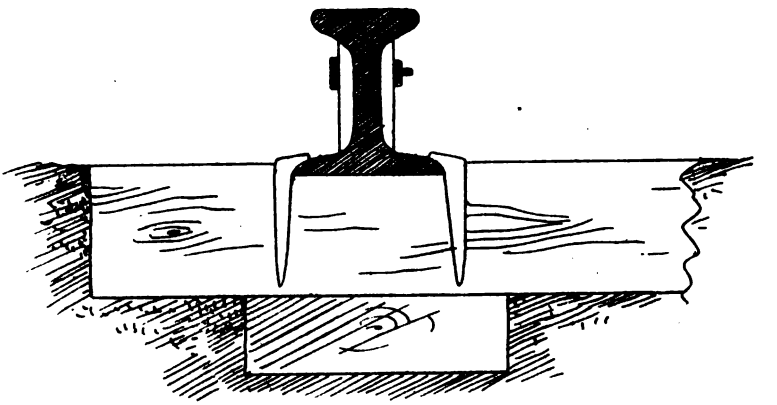
The First American Track.

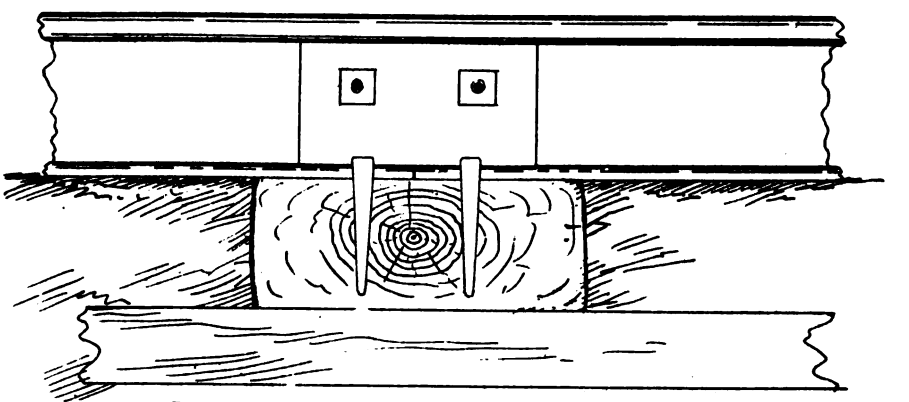
Mr. Francis B. Stevens, of Hoboken, N. J., informs me that in 1835 he was employed by the Camden and Amboy Company to make a profile of the road bed from South Amboy to Camden. At that time there were many places (the longest being a piece 2 miles long, from the wharf at Amboy to Deep Cut), where the Stevens rail was spiked to the cross-tie according to the present practice. This was at first regarded as a temporary expedient on account of the delay in getting stone blocks from Sing Sing. In the meantime it was found that the wood ties were more satisfactory than the stone, and all the stone blocks were replaced by wood ties. Without doubt the Camden and Amboy Railroad was the first in the world to be laid according to the present American standard.

The following extract from a letter written to me by Mr. Ashbel Welch, Past President of this Society, March 18th, 1882, is of great interest:

"The road superstructure for some distance from Bordentown south (1832–33), was laid thus: two white pine 3-inch plank were laid on the ordinary soil, side by side under each rail, white oak blocks, perhaps 6 x 5 inches high and 2 feet long, were laid across those continuous bearers, and the rails were laid on them. Holes 1 inch diameter, 10 inches deep, were bored 4 feet apart in the edges of the plank and filled with coarse salt and plugged up. The salt was all taken up by the wood in a few months. I examined this substructure about four years after it was put down. In all cases within a foot of the salt holes the wood

Digitized by Google





CAMDEN & AMBOY R'Y. STANDARD — 1837.

FIG. 13.

Digitized by Google

Digitized by Google

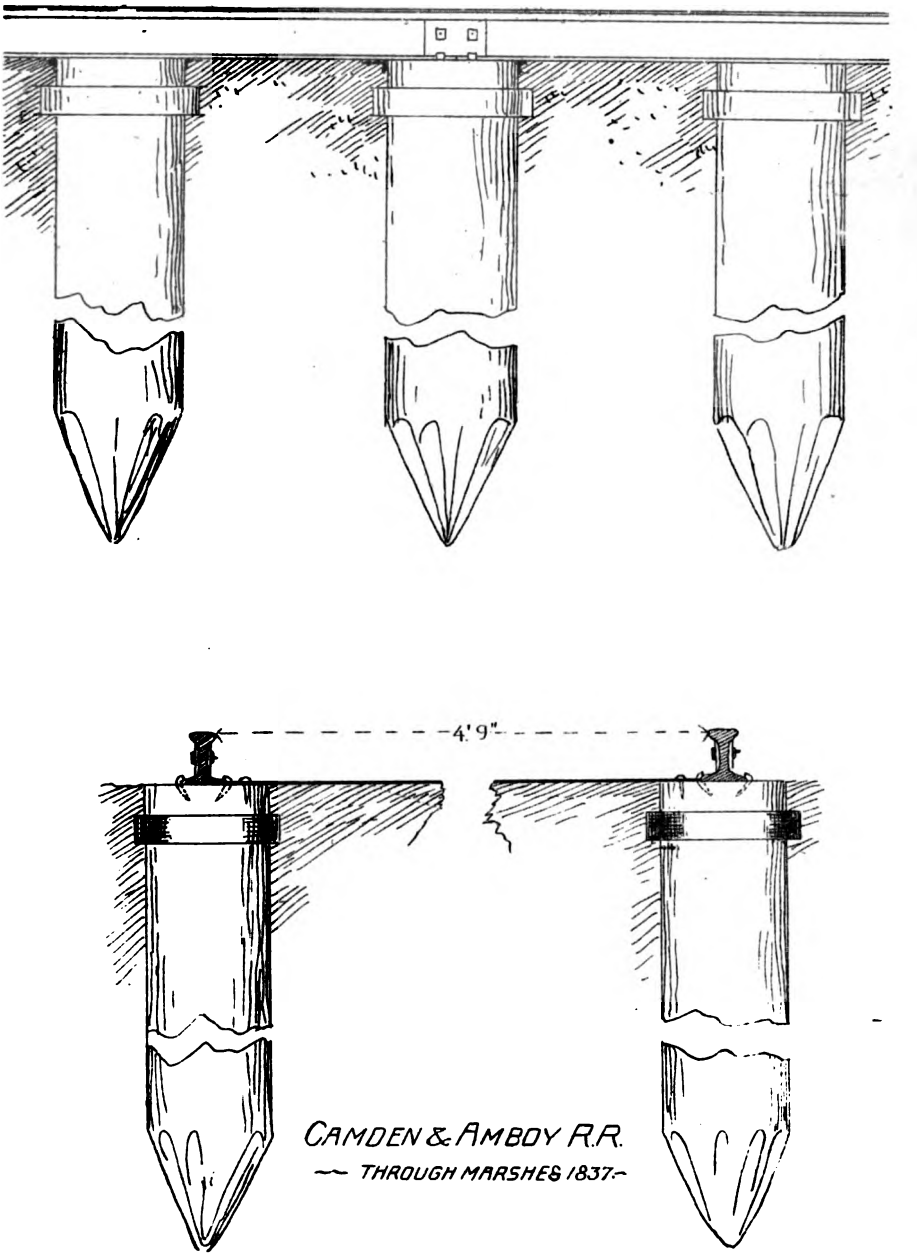
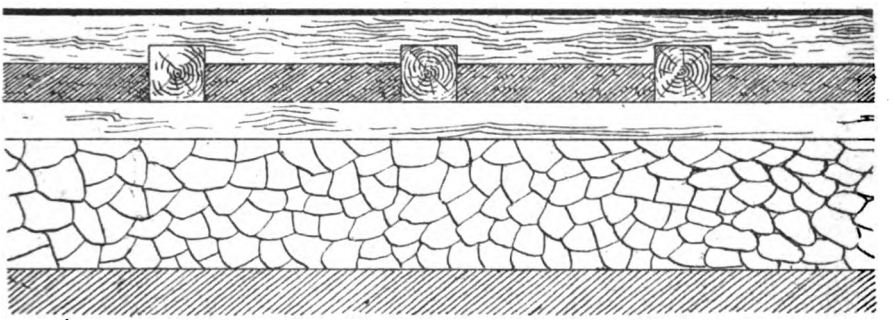
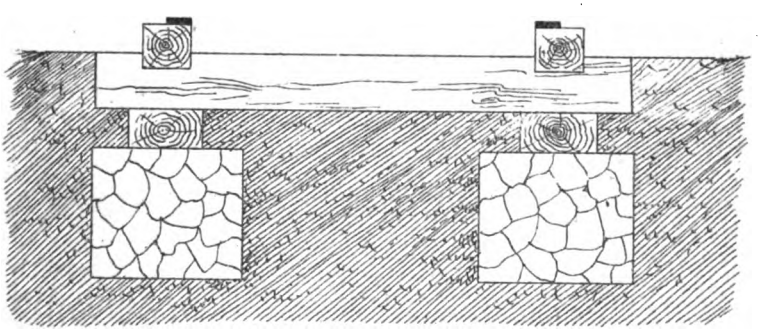


FIG. 14.

Digitized by Google

Digitized by Google





ALBANY & SCHENECTADY R.R.—1837

FIG. 15.

Digitized by Google

Digitized by Google

BALTIMORE & OHIO R.R. ——— 1837.

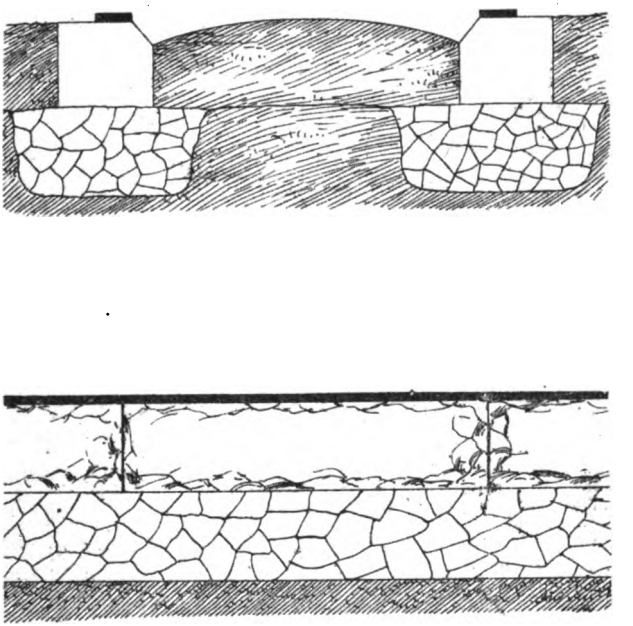


FIG. 16.

Digitized by Google

Digitized by Google

was perfectly sound. The rest of the wood was more or less decayed. The oak blocks were badly seasoned, cracked, and often split to pieces. If, instead of blocks, ties 7 feet long had been used, it would have made a pretty good road. If the salt holes had been 2 feet apart instead of 4 feet, and had been refilled every year, the continuous beams would have lasted many years. The blocks on the bearers were at least in some places salted, but the salt holes only hastened the splitting of the blocks."

In Johnson's "Notes on the Use of Anthacrite," published in 1841, he described tests of cast-iron rails made during that year at Lyman's foundry, near Pottsville, Penn. These rails were designed for colliery railways. They were only 6 feet long. For 3 or 4 inches at each end the rail had a section similar to the Stevens rail; for the remaining 5½ feet the rail was somewhat similar to the English bull-headed rail.

Up to the year 1842, when Congress passed the celebrated high tariff law, all imported iron rails were admitted to the country almost free of duty. The tariff on manufactured iron, agreed upon by Congress, in that year increased the cost of English rails to such an extent that the railways were forced to seriously advocate the erection of American rolling mills for this special purpose.

Rails First Rolled in America.

The first rail mill erected in this country was located at Mount Savage, Allegheny County, Md. The first rail was rolled in the summer of 1844. In honor of that event the Franklin Institute of Philadelphia awarded a medal to the proprietors in October, 1844.

The rail was of the ∩ form, similar to the Evans (British) patent, and the first few hundred tons manufactured were laid on the Baltimore and Ohio Railroad, between Mount Savage and Cumberland. A section of this rail, which weighs 42 pounds to the yard, was presented to the Section of Transportation and Engineering in the U. S. National Museum, by the late Colonel James Randolph, for many years Consulting Engineer of the Baltimore and Ohio Railroad Company. (See Fig. 17, Plate LIII.)

Swank, in his history of iron, states (page 344) "That the first T-rails rolled in this country were made for the Montour Iron Company by Haywood & Snyder, proprietors of the Colliery Iron Works at Pottsville, the work being done at their branch establishment at Danville, Pa.," 1846.

Digitized by Google

Among other early rail mills were the following, with the date when they began to roll rails:

Boston Iron Works	May 6th, 1846
Trenton Iron Works (Cooper & Hewitt, Proprietors)	June, 1846
New England Iron Co, Providence, R. I.	Sept. 1st, 1846
Phœnix Iron Co., Phœnixville, Pa	Nov., 1846

The rapidity with which American capital was diverted in this direction has for the last forty years been one of the great arguments used by the advocates of a high tariff for the protection of American industries.

During the year 1848, a very interesting experiment was tried by the engineer of the Camden and Amboy Railroad. Arrangements were made with Cooper & Hewitt at the Trenton Iron Works to roll a rail 7 inches high with a base $4\frac{1}{2}$ inches wide; 15 miles of the Camden and Amboy road were laid with this rail the following year. The engineer of that company believed that he had at last solved the problem of track construction, inasmuch as this rail gave an admirable opportunity for a strong joint. By experience it was found that this rail was so rigid that it produced so much concussion by the train that the ends soon hammered out, and where the ballasting was imperfect great damage was caused to the rolling stock, consequently the rail was soon after taken up. Much of this old rail found its way to the cities, where it was bought by architects and contractors for building purposes.*

The fact that this rail was rolled successfully resulted in the introduction of the "I" beam for architectural purposes, Cooper & Hewitt having done a large business at the New Jersey Iron Works, at Trenton, in this line, ever since that time. Fig. 18 shows a section of this rail which was laid between Bordentown and Burlington in 1849. (See Fig. 18, Plate LIII.)

Pear Shaped Rails.

The first American T-rails were made of inferior iron, and this was one of the causes that led to the adoption of the section with a pear-shaped head, with which many roads were laid during the next fifteen or twenty years.

We have models of cross-sections of four of the pear-shaped

* Among other places, many of these rails were used for beams in the United States Mint at Philadelphia.

Digitized by Google

rails described in the report of the Railroad Commission of the State of New York for 1855.

Fig. 19. *Rail.* New York and Erie Railroad. Fifty-six pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 20. *Rail.* Buffalo, Corning and New York Railroad. Sixty-two pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 21. *Rail.* New York Central Railroad. Fifty-six pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 22. *Rail.* Saratoga and Schenectady Railroad. Sixty-five pounds to the yard. In use in 1855. (Plate LIII.)

The obtuse angle between the lower side of the head and the stem of the rail made it difficult to apply a splice bar of any kind to advantage, and this fact led to the introduction of the ring joint (one iron ring passing through two slots, one in each stem of adjacent rails and passing around under the base of the rail and held in position by a wedge driven between the ring and the rail-stem). Chairs and other joint fixtures attached entirely to the base of the rail were also experimented with, but generally without satisfaction, judging from the fact that none have survived.

Fig. 23 is a *rail* without base, laid on the Hempstead Branch, Long Island Railroad. Supported by chair; it weighed 40 pounds to the yard, and was in use in 1855. (Plate LIII.)

The difficulty in making good joints with the pear-headed rail was overcome by some of the Engineers by planing away a portion of the head of the rail for a foot or 18 inches from each end. In Fig. 24, Plate LIII, is shown a section of the pear-headed rail, fitted for splice bar, used on the Erie Railroad.

On this rail a cast-iron angle splice containing four bolts and measuring 9 inches in length, was used as early as 1857. On the Pennsylvania Railroad and on the Belvidere-Delaware Railroad, as will be seen in the plate, the rails were constructed with special reference to use of a splice bar almost square at the head and base of the rail as early as 1857.

The ∩ rail with a metal shoe running the full length of the rail to which it was bolted, thus adding to its strength, was used on the Great Western Railway of Canada. (See Fig. 25.)

Fig. 25. *Compound* ∩ *Rail.* Great Western Railroad, Canada West. Sixty-five pounds to the yard. In use in 1855. (Plate LIII.)

Digitized by Google

Compound Rails.

The difficulty in obtaining satisfactory joint fixtures on the American pear-shaped section led to the introduction of the compound rail.

From the same report alluded to above, I have had these five models made of compound rails in use in the State of New York in 1855.

Fig. 26. *Compound Rail.* New York Central Railroad. Seventy-five pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 27. *Compound Rail.* New York Central Railroad. Sixty pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 28. *Compound Rail.* New York Central Railroad. Seventy-five pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 29. *Compound Rail.* Troy Union Railroad. Sixty-five pounds to the yard. In use in 1855. (Plate LIII.)

Fig. 30. *Wide Compound Rail.* Troy Union Railroad. Sixty-five pounds to the yard. In use in 1855. (Plate LIII.)

When the track composed of these compound rails was new, it is described by those who rode upon it as being the finest track of the period, and a large number of rails of different shapes were rolled and laid. No satisfactory nut lock was in use at that time, and as the screw threads wore and traffic became heavier, the different parts of the rails could only be kept together by constant attention, in screwing up the nuts, etc. As the rails laid were of iron, the wear of the inner surface was considerable, so that in a little while the track was badly damaged and the old solid rail had to be substituted.

It is still an unsolved question whether or not (with some improvement in the section, and made of steel and held together with the improved bolt and nut lock), the compound rail may be the rail of the future.

Poor Rails Laid During War Times.

During the next ten years little seems to have been done by American railroad contractors to improve the shape of the rail or joint fixtures; in fact, during the civil war, iron was dear and little rail was rolled, few new railroads being built and repairs to tracks only being made under the gravest necessity. Almost all the forms of rails which were made during these few years were designed by the proprietors of rail mills who naturally adopted such shapes as were easy for them to make, and the railroads, when further delay was dangerous, went into the market and purchased such as were offered at the lowest price, without regard

Digitized by Google

to the shape of the rail or the quality of the iron or whether it was designed for light or heavy traffic.

The Ashbel Welch Rail.

After the close of the war in 1865 it became necessary to re-lay a large percentage of the mileage of almost every railroad. Upon many of the roads some of the same rails were in use with which the roads were originally laid, and the roads between Washington and New York were no exception to the general rule. The late Mr. Ashbel Welch, Past President of this Society, in "A Memoir on Rails," read before the Society, June 10th, 1874 (fifteen years ago), states that "during the year 1865 the task presented itself to me of devising or selecting suitable forms of rails for the system of railroads occupying the central part of the State of New Jersey between Philadelphia and New York, of which I was the executive officer as well as engineer."

The 62-pound Ashbel Welch rail, which was rolled by the Bethlehem Iron Company during the following year, was 4½ inches high, the base being 4 inches and the stem ½ an inch thick; the angle of inclination of bearing surfaces both on the top of the base and bottom of the head being 14 degrees.

A model is presented of the Welch 50-pound and 62-pound rail as originally designed. Substantially this form of rail was adopted by the railroads in the Eastern and Middle States previous to the year 1873, although when the rails were first laid the cross-section was strongly objected to. (See Figs. 31 and 32, Plate LIII.)

Mr. Welch's labors in this direction led to his being considered one of the foremost rail designers in America, and in 1873 he was appointed chairman of a committee by this Society to report on the "Form, Weight, Manufacture and Life of Rails." The other members of the committee were Messrs. M. N. Forney, O. Chanute, and I. M. St. John. The report of that committee, presented at the Annual Convention, June, 1874, was the most exhaustive treatise on the subject of rails published up to that time.

In Mr. Welch's memoir attached to this report, in alluding to his pattern of 1865, he states: "I made one decided mistake in this pattern by not having the outer bottom corners of the head sharp enough; or rather I yielded too much to the feeling against such an unsightly thing as an angle head." The rail proposed by Mr. Chanute

Digitized by Google

in the same report, published fifteen years ago (a model of the cross-section is here), is not dissimilar to the section of standard 66-pound rail, now in use on the Chicago, Burlington and Quincy and several other railroads. (See Fig. 33, Plate LIII.)

Steel Rails Introduced.

I shall not attempt to enter into the history of the introduction of Bessemer steel in America, nor the conflicts in the United States Patent Office, which finally resulted in a compromise and consolidation of the various interests involved.

Taking Swank's "History of Iron in all Ages" again as my authority, I find that "the first steel rails ever made in this country were rolled at the North Chicago Rolling Mill in May, 1865." These were experimental rails, only a few being rolled in the presence of a committee of the American Iron and Steel Association.

The first steel rails ever rolled in the United States upon order in the way of regular business were rolled by the Cambria Iron Company, at Johnstown, Penn., in August, 1867. In no one year during the next five years was more than 40 000 tons of Bessemer steel rails manufactured in the United States.

About 1870–'73 attempts were made by several rail manufacturers to roll rails that should have a steel head and iron web and flange—"steel top rail," it was called. A considerable quantity of this rail was rolled by the Trenton Iron Company, for the New Jersey Division of the Pennsylvania Railroad Company. While this experiment was reasonably successful, the lessened cost of making steel soon afterwards made it practicable to make the whole rail of steel.

I well recollect, while in charge of the Maintenance of Way Accounts of the Amboy Division of the Pennsylvania Railroad in 1873, charging out steel rails at $140 per ton. The output of steel rails, which aggregated 90 000 tons in 1872, increased from year to year, so that in 1882, ten years later, the output reached nearly 1 500 000 tons, the price falling from $140 to $35, one quarter the cost of ten years before.

During the lest ten or twelve years no radical change has been made in the shape of the section of rails laid by first-class railroads. It is true that the constantly increasing weight of the locomotive and of the lading of the freight cars has made it necessary to use heavier rails—the increased metal being put in the head (where the traffic is heavy), or in

Digitized by Google

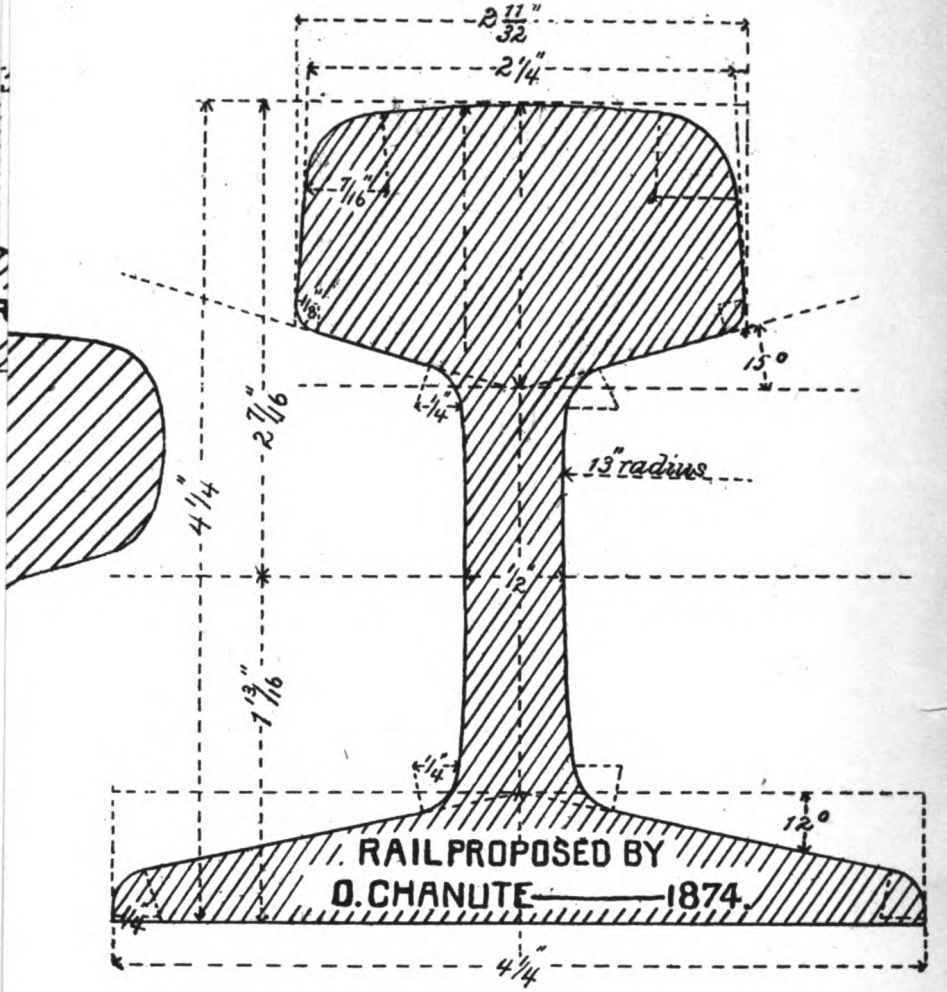


FIG. 33.

Digitized by Google

Digitized by Google

the base (the base of some standard sections being made as wide as 5 or 5½ inches), where the cross ties upon which the rail is laid are of soft wood. The general shape of the rail has, however, been but slightly changed.

Two statements made by Mr. Robert W. Hunt, M. Am. Soc. C. E., in a paper read at a late meeting of Mining Engineers, are of great interest. He asserts that "It is perfectly proper that all the strains to which the proposed rail will be subjected should be considered and calculated, but the ability to resist them will depend quite as much upon the character of the metal as the form of the section," and in another place he states: "The metal commonly known as steel is almost as sensitive to treatment as woman. It will stand any amount of punishment, if administered by judicious hands, and unexpectedly rebel if a less amount is given with indifference. Now, we cannot always control the judgments or rather the actions of irresponsible steel operatives; therefore, sound policy dictates that we give them a section to which they can do the least injury."

That there is an opportunity for some improvement in rail construction would seem to be indicated by the statement made in a paper read by Mr. M. N. Forney, M. Am. Soc. C. E., before the Master Car Builders' Association, in June, 1884. He says: "Probably very few, if any, of the rail sections at present in use would fit the sections of treads and (car wheel) flanges. We have," he continues, "this curious condition of things: the forms of rails have been designed by one set of men, and the wheel treads and flanges by another, apparently without any reference to each other."

Recapitulation.

To recapitulate then, briefly, the steps that have led to the present American system:—the demand for wood to smelt iron reduced the English forests so rapidly that it became necessary to use coal, which was abundant, for fuel: thought was thus directed to the improvement of the methods of transporting coal, which, in order to reduce the cost, was accomplished first by horse-power over wooden tram-ways: and the introduction of the steam pumping engine, which occurred in the meantime, while making a demand for coal fuel, also helped to reduce the cost of mining it, thus making it practicable to use coal in the manufacture of iron, reducing the cost of iron to such a price that it was economical to replace the wooden tracks, first by the cast-iron tram-road,

Digitized by Google

and finally by rolled iron rails, which were made by improved machinery, and upon which the steam locomotive, which had been invented in the meantime and gradually improved, could run rapidly and safely.

In 1830 Robert L. Stevens invented and rolled, under his personal supervision, the first rails ever rolled with a base—designed to be attached to the tie by a spike: before 1835 the Camden and Amboy Railroad, as a temporary expedient at first, laid the first mile of track ever laid in the world practically according to what is known as the American system of to-day: the Stevens rail, as modified and improved by the committee of this Society in 1874, of which Mr. Welch was chairman, determined practically the form and shape of the rail now in general use by prominent American railroads.

Joint Fixtures.

In reviewing the history of the development of the American track, the matter of joint fixture appears to have attracted a very large share of attention; the advocates of each system claiming to have found the ideal fixture, the use of which makes the track as strong at the joint as at any other point of the rail.

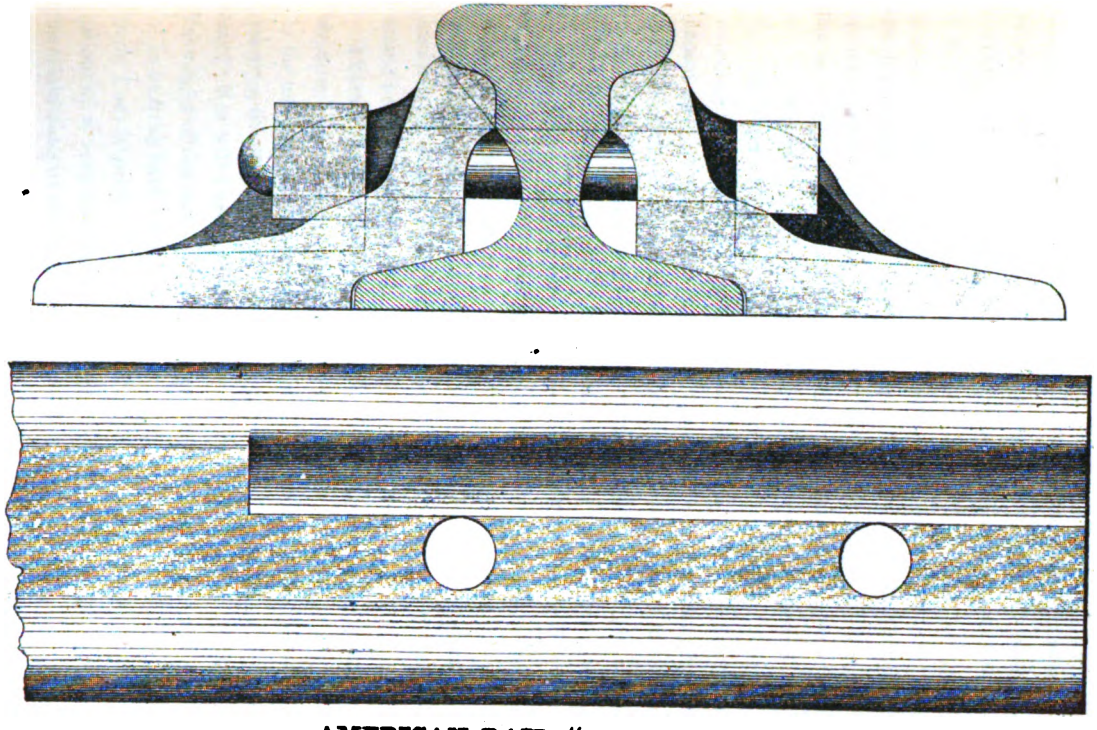
As early as 1857 the angle splice bar (or bracket joint, as it was called) was tried on the Erie Railroad and abandoned (see Fig. 34, Plate LIV). We have known the plain and angle splice bar to be used as short as 10 inches, and have seen it generally increased a few inches at a time, until it has reached 36 inches in length. Hundreds of miles of track have been laid with all the joints "swung." Hundreds of miles (especially since the West Shore Railroad was built) have been laid with the rails supported at the joints.

Each system has its adherents. It occurs to me that when we find the long-looked for fixture which renders the track as strong at the joints as elsewhere, it will make no difference whether the joint is "swung" or supported.

Frogs and Switches.

The scope of this paper will not permit me to take up the question of frogs and switches—further than to call attention to the photograph *fac-simile* of a plate in Mr. William Strickland's report of 1825, to the Pennsylvania Society of Internal Improvement, showing the frogs and switches in use on the colliery roads of England several years before the locomotive was put in regular service (see Fig. 35, Plate LV).

Digitized by Google



AMERICAN RAIL, "ERIE" PATTERN,

with

Ends stamped for Adams Cast-Iron Bracket Splice.

FIG. 34.

1857.

PLATE LIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXII. No. 434
WATKINS ON
AMERICAN RAIL AND TRACK

Digitized by Google

Digitized by Google

English and American Practice.

The traffic on English and American roads, while differing considerably, is similar in many respects.

The admirable paper on this subject,* and the able discussion of it recently published in the *Transactions* of this Society, have placed us in possession of valuable information in regard to the comparative efficiency of the two systems.

The Pennsylvania Railroad's recent experiments with the English bull-headed rail and iron cross-ties, manufactured according to the Webb system, have not, I understand, been very satisfactory, owing to the fact that the heavy rolling stock used on that road causes the metal ties to rise and "float" on the ballast. This would appear to be a defect of the tie rather than of the rail. In riding over several miles of track laid according to this system, near Crewe, in 1886, I was most favorably impressed with its durability (some portions had been in use several years), as well as with the smoothness with which the train rode at high speeds.

In examining the merits of any system, experiments made by laying a few thousand feet of track are not to my mind conclusive.

Is it certain that one system is much better than the other? Perhaps a modification of one or the other would be better than either. Each experiment should be made with the greatest care, nor should conclusions be reached too hastily. When we think of the immense interests involved, it would seem that every movement made in this direction should be encouraged.

I have not ventured (for I am not prepared) to discuss the merits and shortcomings of any of the standard systems of permanent way adopted by the more prominent railroad companies, nor to pretend to answer the question, so often asked: "What is the gravest defect of maintenance of way?" Nor will I attempt to make any prophecy in regard to increasing the weight of the rail or substituting metal for iron ties in the future.

In looking back over the history of the development of the American track, I am impressed with the fact that the experiments and investigations of a very few men have had great influence in bringing about the establishment of the present system.

The prejudices which once existed between American and foreign engineers are rapidly being obliterated. Our parlor cars have been

* "English and American Railroads Compared," by Edward Bates Dorsey. *Transactions*, Vol. XV, January, 1886.

Digitized by Google

experimented with on British roads. An English compound locomotive has recently been imported by a prominent American company for experimental purposes, and a mile of English track has been laid by the same company.

Our journals are full of information in regard to foreign practice. We read of the progress in railroad construction in China, Russia, India and Africa. A single decade of the nineteenth century remains. Upon the doctrine of the "survival of the fittest," may we not hope that the next ten years will decide which system now in existence contains the most elements of promise for the future.

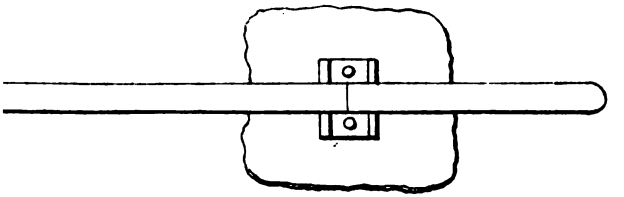
In conclusion, I wish to state that in this paper I have preferred to confine myself to a description of such rails as are represented by original sections, models or drawings in the Section of Transportation and Engineering in the United States National Museum, which is under the control of the Smithsonian Institution. Of this section I have acted as curator for several years, but have only been able to organize active work during the last eighteen months.

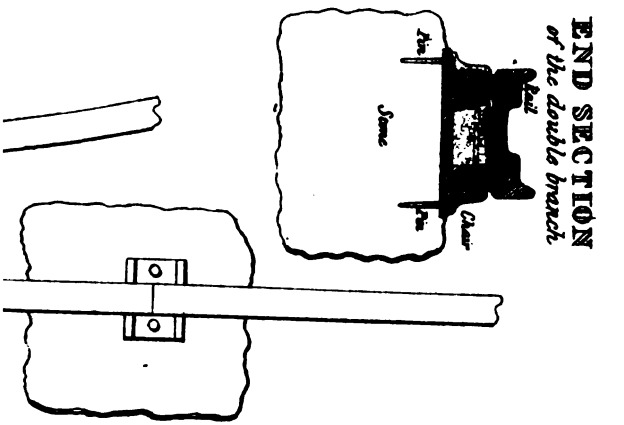
I am fully concious that I have been compelled to overlook many things which are of great historical interest, owing to the fact that our collection is small—only a nucleus in reality.

May I not hope that members of the American Society of Civil Engineers, who have been foremost in so many good works, may each lend a helping hand to increase this collection, so that he who, in the year 1900, writes the history of the birth and development of the American railroad during the nineteenth century may have a much more comprehensive collection to examine and describe than has been my privilege to refer to in preparing this paper.

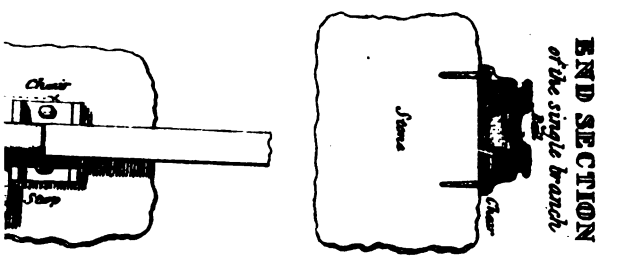
Digitized by Google

PLATE LV.
TRANS. AM. SOC. CIV ENGRS.
VOL. XXXI. No. 434.
WATKINS ON
AMERICAN RAIL AND TRACK.

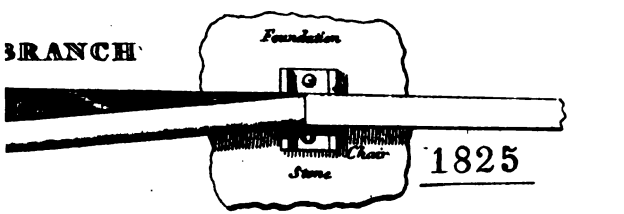




END SECTION
of the double branch



END SECTION
of the single branch



1825

Scale 1 Inch to the Foot.

Digitized by Google

Digitized by Google

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

435.

(Vol. XXII.—April, 1890.)

RAILWAYS OF MEXICO.

By WILLIAM BARCLAY PARSONS, M. Am. Soc. C. E.

The Republic of Mexico is divided into twenty-nine States, with a combined area of 739 700 square miles, and with a population of about 10 500 000, divided as follows, as closely as can be estimated, for there are no reliable statistics :

Whites	2 000 000
Mixed breeds	4 500 000
Indians	4 000 000
	10 500 000

At the present time Mexican Railways aggregate in length about 4 500 miles, with about 750 miles more whose construction is in hand or is about to be commenced at an early date ; not including roads operated by animal power, of which there are quite a number, some of them being of considerable importance, like the line from Vera Cruz to Jalapa, which is 61 miles long.

In Mexico there are practically three trunk or main lines to which all others are feeders or branches. These three are—*First*, the Mexi-

Digitized by Google

can Railway, the oldest and in many respects the most important road in the country, extending from the City of Mexico to Vera Cruz, 263¼ miles, with a branch 29¼ miles, from Apizaco to Puebla. *Second*, the Mexican Central, whose main line, 1 223 miles long, connects the City of Mexico with El Paso, Texas, while branches in addition amount to 595 miles. From Torreon, a station on the Central 517 miles south of El Paso, commences the Mexican International, which running northeasterly 384 miles, strikes the Texas frontier at Eagle Pass, where connection is made with the Southern Pacific. Although the International is a separate organization from the Central, being owned by the Huntington interests, it is, nevertheless, geographically speaking, a feeder to the Central, and forms the principal American connection for that company. In fact, I understand that 65 per cent. of the United States-Mexican traffic passes over the International; the remaining 35 per cent. being divided between the National and the Central line to El Paso. *Third*, the Mexican National Railway, which, starting from a connection with the International and Great Northern at Laredo, Texas, has been completed within the past year to the City of Mexico, 837 miles, while branches bring the total mileage up to 1 217. This line practically parallels the Central, and is a competitor for United States business, although sufficiently removed as not to interfere with local sources of traffic. It has the advantage of providing a line 242 miles shorter to New York than *via* the Central-International, but suffers the great disadvantage of being narrow gauge, necessitating transfer at Laredo.

Of new lines under construction the most important are—*First*, the Monterey and Mexican Gulf, from Monterey, on the National, to Tampico, a port on the Gulf of Mexico, 300 miles. This road is being well built by New York parties, and will form when completed the only low grade line (maximum grade 1.5 per cent.) connecting the Gulf with the interior. It traverses a fine section and will undoubtedly prove successful. *Second*, a branch of the Central from Aguas Calientes to Tampico, 380 miles long, which is just completed. This has been a very expensive piece of work, requiring 4 per cent. grades to make the ascent from the coast. At present the harbor of Tampico is poor, the depth of water on the bar being insufficient. The government, however, has made a concession to the Central Railway to improve it; this done and with two railways, Tampico is likely to become a place of importance.

Digitized by Google

Third, the Interoceanic Railway, which proposes to connect the Atlantic at Vera Cruz with the Pacific at Acapulco, *via* the City of Mexico, 479 miles. About 280 miles of main line and 20 miles of branches are in operation to date. The construction of the Vera Cruz-City of Mexico portion is being actively pushed, and when completed it will form a direct competitor against the Mexican Railway; although its route *via* Jalapa and Puebla is somewhat longer, nevertheless, on account of lighter gradients and much less cost of construction, it will be able to compete with great advantage. The Interoceanic gauge, however, is 3 feet. Both roads are controlled by English capital. *Fourth*, the Mexican Southern Railway, whose construction is about to be commenced, surveys now being in progress. This road is to be owned in England, and is to be built by Messrs. Bowes-Scott, Read, Campbell & Company, the well known firm of contractors, with offices in London and Mexico. The line is to start from Puebla, where it will connect with both the Mexican and Interoceanic Railways, thence *via* Tehuacan and Oaxaca to Tehuantepec. Other branches will be built, which will bring the total mileage up to about 500 miles. This road will be important from the fact that it will carry the Mexican system of railways 300 miles further south, and through a rich though comparatively unknown section of country. Population already in existence, which will be served by it, amounts to about 200 000. It is proposed to use a 3-foot gauge and to work in harmony with the Interoceanic.

In Mexico, railways are built under direct concessions from the government. These concessions confer the right on one or more individuals to construct a line of railway according to the approval of a cabinet officer and an engineer appointed by him. The concession carries either a subsidy of government 6 per cent. bonds at a stipulated rate per kilometer, to be paid to the company as fast as the road is accepted in sections by the government engineer, or a subsidy in cash to be paid annually; at the end of ninety-nine years, the road to pass into the absolute possession of the government, with or without additional payment, according to the terms of the original concession.

Such is in general the form of concession, although of course varied from in certain cases. For instance, to the Interoceanic Railway the government guarantees absolutely for a period of fifteen years, interest at 8 per cent., which period is to be extended to twenty-one years if the road is completed within a term of seven years, while the concession

Digitized by Google

of the Isthmus Railway stipulates that the government shall own the railway immediately on completion and not wait for the usual period of ninety-nine years.

Concessions as a rule either explicitly or implicitly limit the chartering by the government of parallel or competing lines within certain reasonable distances. They also permit the importation of all material required in construction free of duty.

Whether the railways will actually pass into the hands of the government is a matter of dispute even in Mexico, many contending that public interests would not best be served by government control, and that owing to the instability of the Mexican government some default will have been made by it during that period, which will act as a bar to its enforcing its claim. In fact, part default has already been made on most of the existing concessions. But perhaps at the end of ninety-nine years the "President and People of the United States in Congress assembled" will have a voice in the matter, although at present it is not clear that any advantage would accrue to the United States from such a step.

At present Mexican railways are doing better financially than ever before. The following table shows the earnings of the Mexican Railway for three years, the figures being taken from their reports and reduced into United States dollars, in which currency all values in this paper are given unless otherwise stated:

	1888.	1887.	1886.
Gross earnings	$3 863 581	$3 617 561	$3 156 094
Operating expenses	1 307 692	1 267 071	1 261 495
Net earnings	$2 555 889	$2 350 490	$1 894 599
Percentage of operating expenses	34 per cent.	35 per cent.	40 per cent.

Earnings per mile	1888.	1887.	1886.
Gross	$13 186	$12 346	$10 772
Net	8 723	8 022	6 466

These earnings would pay 5 per cent. interest on the following capitalization per mile:

	1888.	1887.	1886.
Per mile	$174 460	$160 440	$129 320

Digitized by Google

The capital account of the Mexican Railway stands as follows:

Ordinary stock	£2 254 720
First preference 8 per cent. stock	2 554 100
Second preference 6 per cent. stock	1 011 960
Six per cent. perpetual debenture stock	2 000 000
Second mortgage stock	249 600
	£8 070 380
Capitalization per mile	£27 600
Or	$135 000

Partly on account of heavy work, but more as the result during construction of bad management and other difficulties, some of them political, it is the most expensive railway in the world.

An examination of the figures shows, first, very high earnings per mile; second, a low proportion of operating expenses, and third, a gratifying increase in business.

The first two of these conditions are due to the very high freight rates charged, and so far to a monopoly of the business. The following is a portion of the tariff on through freight from Vera Cruz to Mexico City, 263 miles, per "ton" of 1 000 kilos—equivalent to 2 204 pounds or substantially a long ton.

	Car Load.	Less than Car Load.
Bricks	$11 25	$15 00
Carpets	.....	30 00
Steel in bars	17 25	22 50
Carcasses of meat	.....	30 00
Oats	18 75	22 50
Crockery and glassware	.....	30 00
Beer in bottles and casks	18 75	22 50
Cocoa	26 25	30 00
Drugs	.....	30 00
Coal and coke	9 00	.....
Iron chains	17 25	22 50
Iron in bars	15 00	18 75
Iron ware	17 25	22 50
Timber in large pieces	11 25	15 00
Butter and lard	22 50	30 00
Condensed milk	.....	30 00
Agricultural machinery	17 25	22 50
Wine in bottles	26 25	30 00
Explosives	.....	33 75
Cloth	17 25	22 50
Furniture	22 50	30 00
Sugar	.....	30 00
Cotton	15 00	18 75

Digitized by Google

The figures are in American dollars, from which we see that the lowest charge (on coal) is at the rate of 3.1 United States cents per ton per mile, while iron ware pays five to six cents, and meat, sugar, drugs, etc., pay ten cents. Local rates from intermediate stations are higher still, the long and short haul of the Interstate Commerce Act not being applicable here.

These rates result from a revision of 20 per cent. made one year since, and have suffered a reduction of 60 per cent. within the last five years. An analysis of operating expenses and earnings shows, taking 1887 as a fair average:

OPERATING EXPENSES IN 1887.

Maintenance of way	$283 563
Conducting transportation	271 352
Motive power	511 286
General expenses	200 870
	$1 267 071

EARNINGS IN 1887.

Passenger	$671 241
Freight	2 894 120
Miscellaneous	52 200
	$3 617 561

The high proportion of motive power is due to the expense of hauling trains up the heavy grades (4 per cent.) and the very high price of fuel, coal costing no less than $10 25 delivered on the engines, although imported in part from England free of duty. General expenses are also high in proportion, but they are swelled by maintenance of London office and expenses incident to exchange, etc. Of freight carried during 1887, foreign goods represented 17.95 per cent. and local 82.05 per cent.; 292 235 tons (1 000 kilos) were hauled, of which pulque, the national beverage, contributed 81 643 tons, while grain stood second in the list with 34 886 tons; these two items together furnishing two-fifths of the whole tonnage.

Digitized by Google

The earnings of the Central for the same three years were—

	1888.	1887.	1886.
Gross earnings	$4 286 203	$3 768 973	$3 026 782
Expenses	2 537 744	2 095 926	1 924 710
Net earnings	$1 748 459	$1 673 047	$1 102 072
Percentage of operating expenses	59.2 per cent.	55.6 per cent.	63.6 per cent.

	1888.	1887.	1886.
Earnings per mile			
Miles operated	1316.4	1235.9	1235.9
Gross per mile	$3 256	$3 049	$2 449
Net per mile	1 328	1 354	892

Proportions	1888.	1887.	1886.
Passengers	23 per cent.	25 per cent.	30 per cent.
Freight	73.5 "	71 "	65 "
Miscellaneous	3.5 "	4 "	5 "

On the Central, National and International Railways, freight rates, while not as high as on the Mexican Railway, are high when compared with those received on our own lines.

As a rule, local rates for car load lots are about seven, five and four cents (United States) per ton per mile for the three classes of freight under their classification. For shipments less than car loads, the rates are proportionately higher. The National, owing to its shorter line to competing points, is enabled to make a higher per mile rate than either the Central or International. Its lowest regular per mile rates are between the City of Mexico and Laredo, a distance of 839 miles, and are 4.3, 3.3 and 2.8 cents per ton in car load lots. Through rates from points in the United States are made by adding the American rate to the frontier to the Mexican rate thence to destination. Local passenger rates charged average about seven, five and three and a half cents per mile for first, second and third class respectively.

The practice of Mexican roads seems, therefore, to develop a small tonnage at high rates, rather than a large one at low rates, which latter method has been proved to be the successful one in the United States, and which eventually will be the same in Mexico too; for the earnings of the Mexican Railways are seen to be on the increase, while their rates are decreasing.

In this connection the author quotes from a report by the British Secretary of Legation to Mexico, Sir Francis Denys, from which some other information contained in this paper has been obtained.

"In spite of this reduction, the traffic returns of the Mexican Railway show an increase, which would lead one to suppose that the local and

Digitized by Google

through traffic would have been more quickly developed in the past, if the system of high tariffs, almost prohibitive in case of some goods, had not always prevailed.

"Reduction in rates is needed for heavy goods, such as iron castings, plates, chains, beams, etc.—in short, on all those articles whose value does not exceed £7 per ton, or thereabouts, and which evidently cannot bear a freight charge of £6 per ton in addition to duties and sea carriage.

"Through and foreign traffic on this, as well as on other lines in Mexico, are granted facilities which are not extended to local traffic, and shippers of goods at either end of the railway can obtain a better rate than those sending goods from intermediate stations. Consequently, it is often cheaper to send goods over the roads by mules and donkeys than to use the railroad, and this is done to an enormous extent throughout the country served by the various trunk lines, it being not an unusual occurrence to see loaded trains of both animals following the roads or tracks skirting the railroads—traffic which is lost to the railway.

"Goods can also often be carried past their destination and back again at less cost than if they were sent direct in the first instance."

There are two causes which seem to retard the development of Mexican railways: First, the very high railway rates; Second, the lack of a good cheap fuel. To any one accustomed to the rapidity with which development follows a railway in the United States, the apparently small effect produced by a railway on the tributary country in Mexico is very striking.

Even along the line of the Mexican Railway, the oldest in the country, farmers still scratch (literally) the ground with a small wooden plow, while a piece of timber drawn by oxen serves as a harrow. Such sights are seen not only at a distance from the railway, but even from the very car windows, while on several occasions I saw men reaping with sickles. The large haciendas or farms are furnished with a circular tiled floor where wheat is thrashed by mules treading on it, and sugar is still pressed from the cane by passing it through wooden rolls, although iron rolls are now coming into use. The people themselves are naturally unprogressive, and are possessed of little energy, no ambition, and have scarcely any desire and little money to provide for themselves beyond their immediate and very simple wants; and, therefore, require low rates and extraordinary facilities to encourage them to change their customs, habits and tastes, to which as a nation they have become firmly attached through the course of several centuries.

The second retarding cause is the lack of a good and cheap fuel. At present the railways burn wood whenever it can be obtained, but the

Digitized by Google

PLATE LVI.

Vol. XXII, p. 247.

Digitized by Google

Digitized by Google

PLATE LVII.

Vol. XXII, p. 258.

Digitized by Google

Digitized by Google

PLATE LVIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 436.
HARRIS ON
FERRY STEAMER SOLANO.

Digitized by Google

Digitized by Google

supply not being great or widely distributed at best, is yearly growing less. In Northern Mexico coal is imported from the Indian Territory, and is known by the same name as is usually applied to it through Texas and the West, viz.: "Nation coal." In quality this coal is only fair, carrying a heavy proportion of ash, but such as it is, it costs at Monterey $8 50 to $9 per ton at wholesale. Coal has been discovered and mines opened at Hondo, on the International, which are the only mines worked on a commercial scale in Mexico. An analysis of this coal I could not obtain, but in quality it was said to be not better than "Nation coal." It sells at the mine for $4 49 Mexican or about $3 37 American. On account of freight charges it costs in the City of Mexico $11 50, of which the mine gets $3 and the railway the rest. This Mexico rate, however, is "special," being at the extraordinarily low figure of $\frac{8}{10}$ cent per ton per mile.

Coal at intermediate points sells much higher—thus at Queretaro, 245½ miles less haul, it costs $12 30—another instance of Mexican rate making.

The following table gives the price of coal at several important points, as shown by the company's published price list, and their respective distances from the mines:

Chihuahua	$9 52	624 miles.
Jimenez	7 96	478 "
Zacatecas	9 24	582 "
Aguas Calientes	10 05	657 "
Sila	11 39	753 "
Queretaro	12 30	868 "
Tlalnepantla	13 85	1 031 "

The freight charges are substantially 1 cent per ton per mile. On the three Northern lines the rate on coal is limited by the concessions to 1.9 cents per ton per mile.

The Mexican Railway imports annually from England "patent" fuel, coal dust pressed into bricks and cemented with a preparation of pitch. It is found that on the heavy grades, when the engines are being worked with strong exhaust, there is less loss from unconsumed coal in small pieces being drawn from the fire boxes through the tubes with the patent fuel than when ordinary soft coal is used.

In point of physical conditions and details of operation railways in Mexico differ widely. The Central, International and National closely re-

Digitized by Google

semble in their construction railways in our Western States. Rails are light, 45 to 56 pounds, laid usually on Texas pine ties, although some of the local timber is sometimes used, like Mesquite, which is a very durable wood.

Track is not ballasted, except with ordinary soil, although during the dry season it is usually in fair line and surface, at least sufficiently so for the quality and speed of the traffic.

The most noticeable defect of the permanent way is design of the bridge floors, which is invariably poor, the floors being composed of ties of ordinary length (8 feet), set rather far apart and held at the ends by a light wooden guard rail.

The National Railway is now doing, however, considerable good work in replacing wooden trestles with arch or box culverts, where the height of embankment permits, and elsewhere in erecting iron trusses on masonry. These bridges are being built for standard gauge requirements.

The buildings are usually of wood, although the National is beginning to use stone. The equipment is of American manufacture and plan, except that there are three classes for passengers. The first class resembles the ordinary American first-class coach with plush seats; the second-class has wicker covered seats, while the third-class cars are quite cheaply fitted up, with seats running longitudinally, including a bench down the center. The majority of the passenger travel is third-class, and usually the first and second-class share one coach, being separated by a cross partition. The engines are, of course, American, of the consolidation and mogul type. On the International the grades are light, not exceeding 1 per cent., while on the National and Central they run as high as 3 and 4 per cent.

The personnel of these roads is American as far as engine men, conductors and station agents are concerned, but few natives being employed in these positions. Among the firemen and brakemen Mexicans divide honors with Americans, while they provide all the lower classes of labor, such as trackmen, etc.

Enginemen are paid $100 to $140 per month, conductors $85 to $125, and brakemen $50 to $60. Expenses of living are, however, high. Common laborers receive fifty cents to $1 per diem.

Construction prices on a piece of work now in hand were eighty-five cents for solid rock, thirty-three cents for loose rock, and fifteen cents for

Digitized by Google

earth per cubic meter, road-bed measurement. Train service is fairly regular, but this is not a difficult result to accomplish, as scheduled speed is slow. On the Central, passenger trains require sixty hours to run 1 224 miles, an average of 20 miles per hour; on the International, forty-four hours are consumed in the through run, an average of 19 miles, while on the National the average speed is less than 18 miles.

But the Mexican Railway presents the most interesting subject for study. As stated before, this road carries the heaviest tonnage of any line in Mexico, and as it uses 4 per cent. gradients, the permanent way requires considerable stability, although, with the exception of the use of metal ties, its condition in this respect is hardly up to its reputation. The track was originally laid with bull head rails on chairs, but these did not prove successful, and are now entirely displaced by the flat-footed rail. Some of the old rails are still to be seen in side tracks.

The present rails weigh 62 pounds, except that 82-pound rails are used on the ascent between Vera Cruz and Esperanza, where 7 500 feet are overcome in 111 miles. As to joints there is no rule, fish plates and four and six bolt angle plates being used.

Ballast is of sand and gravel, and in places on the heavy gradient an attempt at broken stone is made. The most interesting thing, however, in connection with the track, is the use of metal cross-ties. Wooden ties, expensive in first cost, were found to last only four to five years, so that metal ties were adopted for economy. The ties are of mild steel and weigh 110 pounds each. The rail is held by clamps punched up from the tie, and secured by a key weighing 1 pound, so that tie and two keys weigh about 112 pounds. In section they are channel shaped, with a stiffening ridge down the center. The ends are splayed and turned down, so as to retain the ballast, while the tie bends upward towards the ends, so as to give the rails a cant. Their cost varies with the price of steel, but is now about five shillings, say $1 25, F. O. B. English ports. Ballast is laid flush with the top of the tie. The results of experience have been most gratifying.

Track remains in better surface and line than when wooden ties were used, and certainly the track in best condition to-day is that portion where metal ties are laid. Practically it has been found that the track thus laid requires so much less work that section gangs on metal tie sections have been reduced about one-half. The Mexican

Digitized by Google

Railway is therefore laying these steel ties as fast as possible, there being enough on hand to finish about 150 miles of road.

The bridges are decidedly English in design, many of the shorter spans taking box girders and all differing in every detail from American practice. The floors are the weakest portion, some of the structures having none at all. The rail frequently rests on a small wooden stringer, and even when a regular floor is used it consists of short ties, with no guard rails, unless the bridge be on a curve, when one iron rail is laid next and close to the inside rail. But this rail seems to be laid more on account of the curve than of the bridge. Station buildings and grounds are models of neatness and durability, and offer, especially in the latter characteristic, some good examples for American roads to follow. The buildings are always of stone, of good, serviceable, though not elaborate or decorative design.

The platforms are paved and are raised to the level of the car platforms. The station grounds are inclosed with a high stone wall, heavy wooden gates closing at night the track entrances as well as the driveways. The dining stations differ from nearly all others in Mexico, by providing a very fair meal.

The equipment also presents many features which arrest the attention. The ordinary locomotives, of the mogul type, built at the Baldwin Works, have no bells, so that all signals are given, English fashion, with the whistle. On the heavy gradients double-ended Fairlie engines, built at Sheffield, Eng., are substituted for the Baldwins. The passenger equipment is of English make, and is a poor imitation of an American coach. The cars are short, being about 45 feet long. The sides are of corrugated iron, and the ventilated clere-story is omitted, the roof being slightly curved, through which the lamps are lowered from above, as in Europe. The bell-cord is accessible only on the platform. The cars, however, are quite comfortable, in that seats are set far apart, giving plenty of space. The coupling and brake arrangements are, on the other hand, very defective. The old-fashioned link and pin, with plenty of slack, is in use on all cars, as are also hand-brakes, an automatic brake not being adopted. In this age of railway improvement the safety of a passenger train on 4 per cent. gradients should not depend on the faithfulness and alertness of brakemen. One singular defect in designing the coaches was in the smallness of the windows, and then, in making bad worse, by dividing the panes vertically

Digitized by Google

and fastening the parts with a broad wooden slat. The passengers should be given every opportunity to observe the magnificent scenery, which rivals, if not exceeds, that of any other railway in the world. As is usual everywhere in Mexico, smoking is permitted in all cars. There is no parlor car service. The freight equipment, built both in England and in the United States, is on the American plan. All wheels under freight, as well as passenger cars, have steel tires and wrought-iron spokes, and all draw-bars are of the latter material. The telegraph poles are of cast-iron, neatly designed, while the water tanks are of riveted plate iron on masonry foundations.

In details of operation the Mexican Railway differs strongly from the roads previously described. Train men and other employees are, almost without exception, Mexican, and English is spoken but little along the line. The official time-card presents some novelties, especially in that it gives in parallel columns the arrival and departure of trains at every station.

There are twenty-six stops between Mexico and Vera Cruz, and in the thirteen hours required to make the run of 263 miles, two hours seven minutes are spent in such stops, of which sixteen are of two minutes (the shortest) each, twenty-seven of three minutes, one of four minutes, six of five minutes, one of twenty minutes at Junction point, and one of thirty minutes, stop for dinner. An American superintendent would think that time was unnecessarily lost in stops, but time is the last thing to be considered in Mexico. One point on all railways in this country, but especially on the Mexican, which strikes the American traveler as amusing, is the strong military escort accompanying trains. Every passenger train on the Mexican Railway is guarded by a detachment of a score or more of soldiers, who ride in a special car, while on every station platform are to be seen guards uniformed and armed with a carbine and cumbrous sabre, and in number according to the importance of the place.

The Interoceanic Railway, while following American practice as to road-bed, leans towards European equipment more even than the Mexican Railway. Its passenger cars have compartments, while the locomotives are of the regular English pattern.

One great physical drawback which the railways have imposed on themselves is the difference in gauge. The National was built 3-foot gauge because its promotors were those who had so constructed the

Digitized by Google

Denver Rio Grande. At the time it was built the St. Louis, Arkansas and Texas and several other roads through Illinois, Indiana, Ohio and Pennsylvania were narrow gauge, and part of the project was to construct a great transcontinental and international narrow gauge system.

But the Denver Rio Grande is being changed to standard, and all the other roads have changed, so that the Mexican National now stands alone. The Interoceanic and some of the smaller lines are narrow gauge, and the Mexican Southern proposes to be the same.

The day will come, however, when these roads will have to do as the others in the United States have done, and make their gauge conform to that of the whole North American system.

Digitized by Google

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

436.

(Vol. XXII.—April, 1890.)

THE RAILROAD FERRY STEAMER "SOLANO."

By ROBERT L. HARRIS, M. Am. Soc. C. E.

During a journey in 1885 to and from my old home in California, I saw much to interest me socially and professionally. Among the objects of engineering development was the Railroad Ferry Steamer "Solano," which is supposed to be the largest in the world and which belongs to the Central Pacific Railway Company, and is in use at the Straits of Carquinez.

Perhaps this subject has peculiar interest to me, in that memory reverts to the "fifties," when the great boat "Maryland" ferried the Philadelphia, Wilmington and Baltimore Railroad passengers across the Susquehanna River at Havre de Grace. This boat now runs on the East River in connection with the New York and New England Railroad and the Pennsylvania Railroad. The capacity of its two tracks is fourteen freight cars, which may be borne in mind as a familiar standard for comparison.

That ferry was where a bridge was then considered too stupendous an undertaking to be seriously thought of, and where early in the next decade George A. Parker, upon the completion of the wooden spans,

Digitized by Google

placed this legend upon his bridge: "The work of one thousand men for three years." (This next decade was, however, the Youthful Giant period of the United States, when grand work was done, and for the first time in its history our people became familiar with immense combinations of men, material and money.)

Memory has thus brought me forward to the "sixties," when a "grand" ferry-boat, the "Louise," with its new contrivance of compound engines (and more levers than a man could jump to), was built to run between San Franscisco and the modest little "San Franscisco and Oakland Railroad," 4 miles in length, for which I obtained the first financial backing and was engineer, and which we then truly prophesied would grow to be the terminus of the great Transcontinental Railway, before such a thing existed except in the minds of enterprising, dauntless men.

Still later, memory brings me to the "seventies," when as Chief Engineer of the Chicago and Canada Southern Railway, a trial trip was made on the Detroit River with the large railroad ferry-boat the "Transfer," with three tracks to convey at once twenty-one freight cars between Amherstburg in Her Majesty's Dominion and Grosse Isle in the United States. Let me say, in passing, that this is Grosse Isle (French), not "Goose Ile," as its first post-marking stamps actually came from the Washington General Post Office in 1872.

We now reach the "eighties," and on the Atlantic Coast I note that the ferry across the mouth of Chesapeake Bay, on the route of the New York, Philadelphia and Norfolk Railroad, is probably the longest regular railway ferry in the world (24 miles, time card two hours from Cape Charles to Old Point Comfort, or 36 miles, three hours, to Norfolk). Its fine ferry steamer, the "Cape Charles," which was completed a year ago, attained at its trial trip a mean rate of speed of 18½ statute miles per hour with steam pressure of 55 to 60 pounds. This boat is arranged for the transfer of four passenger cars. Its dimensions are given as follows:

Length over all	259 feet 6 inches.
Length on 6 feet water line	252 "
Beam, moulded	36 "
Beam over all	63 " 6 "
Depth from top of keel to top of iron beams at lowest place	14 "
Displacement at draft of 7 feet	794 gross tons.

Digitized by Google

The paddle wheels (feathering) are 24½ feet diameter. Each wheel has twelve floats which are 9½ feet long and 40 inches wide. The motive power is one surface-condensing beam engine with cylinder 54 inches in diameter and stroke of piston of 11 feet.

And now, in the "eighties," I find that the Pacific Coast has the largest, finest, most complete of all railroad ferry steamers yet launched, the "Solano," which so interested me that I asked for details, which were cheerfully given by my friend, Mr. Arthur Brown, subscriber to the building fund of this Society, whom I formerly knew as in charge, during the construction, of all wooden structures built upon the line of the Central Pacific Railway, many of which required rapidity in construction as well as originality and boldness of design; the history of which, if written, would alone form a grand chapter in the Civil Engineering of the past fifty years. The description kindly furnished by Mr. Brown is as follows:

DESCRIPTION OF THE CAR FERRY STEAMER "SOLANO" AND THE SLIP AT PORT COSTA, NORTHERN RAILWAY, CALIFORNIA.

By Arthur Brown, Superintendent Bridges and Buildings Department, Central Pacific Railroad.

West Oakland, Cal.

The ferry steamer "Solano" was built for the purpose of carrying a full train of cars (viz., forty-eight freight or twenty-four passenger cars with engine) across the Straits of Carquinez, on the line of the Northern Railway, a leased line of the Central Pacific Railroad. The current in these straits runs at times 8 miles per hour. The range of tide is 9 feet. The following are the general features and dimensions:

Length on deck	424	feet.		
" of hull (between perpendiculars)	406	"		
Height at center	18	"	5	inches.
" at ends	15	"	10	"
Moulded beam	64	"		
Width over guards	116	"		
Camber (reverse shear)	2	"	6	"
Light draught	5	"		
Loaded draught	6	"	6	"
Registered tonnage	3 541 $\frac{2}{100}$			

Propelled by two vertical beam engines, placed on the center of boat, 8 feet fore and aft of midships, making the distance from center to

Digitized by Google

center of shafts 16 feet. Each engine drives one wheel, and works independently of the other.

Engines:	
Cylinders	60-inch bore.
Stroke	11 feet.
Wheels:	
Diameter	30 "
No. of buckets	24 each wheel.
Width of buckets	30 inches.
Face " "	17 feet.
Boilers:	
Number of	8
Length	28 feet.
Diameter of shell	7 "
Number of tubes	143
Diameter " "	4 inches.
Length " "	16 feet.
Heating surface, total	19 640 square feet.
Grate " "	288 " "
Capacity	2 000 horse-power.

The boilers are of steel. They are placed in pairs on the guards fore and aft of the paddle wheels, and are so connected with the engines that one or all may be used at pleasure.

The boat is stiffened longitudinally by four wooden Pratt trusses (vertical posts and diagonal rods), one under the center of each track.* The members of these trusses are proportioned to stand the heaviest loads which can come upon them. In calculating the strains, the difference between the sum of the weights at each panel, and the displacement for the panel length was assumed as the maximum force acting at that panel. For several panels at the center and ends of the boat the weights exceed the displacements, but, between, the reverse is the case. A separate calculation for displacements, etc., was made for each position of the moving loads.

Success in handling a boat of such dimensions in an 8-mile current is due principally to the use of balanced rudders in connection with hydraulic steering apparatus, and to independent engines. There are four balanced rudders coupled together at each end of the boat; at the forward end they are held in position by the pin, which is removed at the stern.

The axis of the slip on each side of the straits coincides nearly with the direction of the current, pointing on the Port Costa side up, and on the Benicia side down stream, so as to facilitate entering the slips. The distance across is one mile; the whole time consumed in transit is nine minutes, including starting and stopping.

The "Solano" has been running four years without trouble, and is handled with great ease. The variation of tide rendered necessary the use

* Shown in cross-section as between each track at midships; better shown in plan.

Digitized by Google

of a hinged apron, *A* (see Plates LIX and LXI), supported in part by a wholly submerged pontoon, *B*, and in part by the counterpoises *C*. The apron is 100 feet long by 44½ feet wide, and carries four tracks, corresponding with those on the boat. Its weight is about 154 tons. The pontoon displaces about 65 tons, and the counterpoises about 12½ tons each, the remainder being carried by the hinge H. When in use the apron rests upon the end of the boat in a recess into which it fits with a little play, so that the wharf and boat act as abutments for the moving load.

The apron is composed of five longitudinal bow-string combination wood and iron trusses, from which are hung floor beams carrying the track stringers.

When the boat has entered the slip and is in position to receive or discharge a train, the counterpoises are raised by hydraulic power; leaving 25 tons of the apron unbalanced, this sinks the pontoon, and when the end of the apron reaches its place on the boat it is securely latched down; the counterpoises are then released; the apron and boat are now free to rise and fall with the tide, and the apron is ready to rise from its seat as soon as unlatched from the boat. The boat is held up to the apron by means of two mooring rods, which run the entire length of the apron, are hinged at *H*, and bolted back, as shown, to the piling. The rods are connected with the boat by means of links and tightening levers. The wire rope *g g* connecting the counterpoises with the apron on each side is double and continuous; it passes half around equalizing sheaves, one at the end of the supporting beam of the apron, and one at the counterpoise.

From the top of the boxes containing the counterpoises the chains *K* pass partly around the sheaves *l l*, and the sheave *m* in the crosshead *J* of the hydraulic lift *O*, and are secured at *n*. From the bottom of the counterpoise the chain *K'* passes similarly around *l' l'* and *m'*, and is secured at *n'* (Plate LXI). The lift is supplied from the accumulator *G*, into which fresh water is pumped by hand from the tank *R* by means of the pump *I*. It is necessary to have a man at each slip, and he has sufficient time to attend to pumping, so that power is obtained at no expense. The distribution valve of the lift *T*, shown in detail (Plate LXI), is so arranged that when the power is cut off the ends of the cylinder are connected, so that the piston may be drawn either way and the apron rise and fall with the tide, the water in the cylinder merely circulating around. When the counterpoise rises or falls it pulls the piston of the lift toward one or the other end of the cylinder by means of the chain *K* or *K'*. The stroke of the piston is sufficient to allow this change of position and leave enough to handle the apron, take up lost motion, and allow clearance, so that, whatever may be the position of the tide, the cylinder is always filled with water, and the piston connected and ready to act. There is a lift on each side, but only one accumulator, one tank, and one set of pumps.

Digitized by Google

The journal of the hinge *H* is cylindrical, the axis being at the base of the rail, so that there is no longitudinal motion of the rail.

The position of the wharf relatively to the boat is such that the angle between high and low tide is divided by the apron. Besides this, at extreme tides, the angles are filled with steel wedges, hinged one at each end of each rail on wharf, apron and boat. These, when not in use, lie to one side of the rail; when it is necessary to use them they are simply turned over on the rail. These wedges are of such dimensions that they divide the angle at extreme tides into three other angles nearly equal, enabling trains to pass without inconvenience.

Plate LXII shows more in detail the steering gear and location of engines; referring to the plan, *EE* designate the location of the engines, and *A*2, *A*3 the engine shafts already referred to; *P* the hydraulic pumps which are arranged in pairs; diameter of water cylinders, 4 inches; steam cylinders, 12 inches, each 12 inches stroke. The discharge from each water cylinder is connected with the accumulator *A*, which consists of a tank 30 inches diameter by 8 feet in length, closed on top, kept partially filled with water. The air over the water is kept at a constant pressure by the pumps. In the connection between the accumulator and pressure pipes is placed a weighted valve; this is connected with the engine throttle, and changes the set of the throttle automatically so as to prevent variations of pressure in the hydraulic cylinders. Pressure pipes lead from the accumulator tank to the three way valves located in the hold of the boat under the pilot houses; these valves are shown more fully in detail. It will be thus seen that by moving the valve the pressure may be thrown on either hydraulic piston as may be desired, at the same time changing the exhaust from either cylinder to the tank *T*, to which the suction pipes from both pump cylinders are connected.

The hydraulic cylinders have each a stroke of 14 feet and bore of 5½ inches. There is a pair of hydraulic cylinders connecting with each set of balanced rudders, shown on plan, and more fully shown in detail.

The pressure pipes connect with the head of each hydraulic cylinder; the pressure is thrown from one to the other by means of the three-way valve, and the rudders are held in position by closing these valves. As one piston advances the other recedes.

The valves are worked, from either pilot house, by valve rods leading to valves.

The tillers are connected to steering wheels by means of chains and rods running over fair leaders, and to the steering wheels in the usual way.

The steering wheels turn with the motion of the tillers the same as though they were used directly. The object of this is that, in case of derangement of hydraulic apparatus, the boat can be steered in the usual way.

Digitized by Google

A peculiar feature of the hydraulic cylinders is that should it become necessary to steer by hand, the piston rods, which are a continuation of rods connecting with the tillers, slide through the pistons, diminishing the power required to turn the steering wheels by the amount of friction necessary to move the pistons and also the water through the pipes, the water being allowed to run from one cylinder to the other through the straight passage in the valve, until the pistons are at the head of the cylinder.

By referring to the detail of balanced rudders it will be seen that the four rudders on either end of the boat are connected by coupling rods; the coupling rod connecting the intermediate rudders of the four (it will be seen from end view) is bent upward so as to run close under the fan-tail, and has a hole to receive a pin which holds the rudders in line with the keel.

Plate LXII shows the location of the trusses in plan, and the division of the hold of the boat into eleven water-tight compartments by transverse bulkheads; these also add lateral stiffness to the frame.

In order to appreciate fully the location where the ferry steamer "Solano" is used, it is necessary to see it; next to that, one should have charts; failing which, I will say that the summer traveler from the East, heated and tired by his six days' ride, is suddenly conscious, when 50 miles from San Francisco, of a delicious coolness, caused by the trade-wind from the Pacific Ocean. At about 30 miles from the terminus there is a stop of a minute, a start followed by a pause of about five minutes, when he feels that in some way his motion has changed, and, going to the car platform, discovers that while he is still on the cars, the entire train is on a boat, which is crossing the Straits of Carquinez.

The Straits of Carquinez is a narrowing of what is really an arm of the sea, which, after entering through the narrow "Golden Gate" (so first named by Gen. John C. Fremont), extends in the Bay of San Francisco southerly over 30 miles (extreme width 11 miles) and northerly 12 miles (extreme width 9 miles). It then extends through San Pablo Bay about 13 miles (width 12 miles); thence through the Straits of Carquinez 5 miles, and terminates in Suisun Bay with its area of 11 miles by 5 miles.

This (Suisun) bay receives the drainage of the larger portion of the great State of California through its main rivers the San Joaquin and the Sacramento. I may add that, at a low stage of water in the river, the tide is noticeable at Sacramento, nearly 60 miles in a direct line above the Straits of Carquinez, or by the river navigation 125 miles from San Francisco.

Digitized by Google

The width of the Straits is about 1 mile; the shores are bold; soundings are over 100 feet; the current is dependent on the state of the tide, but is said to be at times 8 miles per hour, and every summer afternoon a gale of cold air rushes through this opening in the coast ranges to diffuse itself in the heated interior of the State, which makes life there livable. My remembrance is that the difference in temperature of a summer day between San Francisco and Sacramento is from 30 to 50 degrees.

Members on our Atlantic Coast may be interested in the following characteristics of the tides of San Francisco:

"There is a large tide and a small tide, and the tides are known as high and higher high (or big high and little high), low and lower low; the lower low occurs after the higher high tide.

"The mean rise and fall of tide is..... 3.6 feet.
" " " spring tide..... 4.3 "
" " " neap tide..... 2.8 "

"The average difference between higher high and lower low water of the same day is 5.2 feet. The greatest observed difference between the two low waters of one day was 5.3 feet, and the greatest difference between the higher high and lower low water of one day was 8.5 feet." My recollection of what I considered the difference in tide is 8.3 feet.

To return to the ferriage: the enjoyment of the breeze is enhanced by the grand views of the beautiful bays and their surrounding hills and mountains which lie in ranges parallel to the coast, among which are seen the peaks of Mount Diablo near by and 30 miles east of San Francisco, and Mount Tamalpais, 20 miles north of San Francisco, each having an altitude of about 3 000 feet, yet in different ranges on opposite sides of this noble fiord.

The traveler hears but little noise and feels but little jar, for although the engines are powerful the hull is very strong and stiff; he sees no coal dust nor dirt, for the two or three firemen have but to tend petroleum and steam jets (a California friend who has just arrived corrects me; he says "one or two firemen in handsome business suits and white unsoiled shirts tending taps"). This mode of heating has proved so successful on the "Solano" that it is being introduced on the railroad ferry boats between the terminus at Oakland and the City of San Francisco, which ferriage is about 4 miles.

Before the traveler comprehends the neatness or has time to question the economy of the fuel, before he appreciates the boat or fully enjoys

Digitized by Google

the scenery, he finds himself again rushed by the locomotive over the steel rails.

I passed back and forth at this ferry several times; at each time the perfection of its operation challenged admiration.

A passenger train of, say, ten cars comes to the slip. During its pause of a minute, the train has been uncoupled in the middle and a switching engine has been attached to its rear; the regular engine proceeds with the forward half of the train on to the boat, the switching engine pushes the rear half on another track of the boat, the steamer is cast off with the entire train and both engines as cargo, and yet only two of the four tracks are occupied, for the capacity of the "Solano" is twenty-four passenger cars or forty-eight freight cars with locomotive.

The landing of the train is as simple as its embarkation, and is made in the same manner. The total detention of a passenger train at both sides of the Straits, by reason of the ferry, seldom amounts to fifteen minutes; the time card shows for the station of Benicia on the north and Port Costa on the south of the Straits, twenty-five minutes total difference, including embarkation, ferriage, landing and station stops.

This ferriage forms a most pleasant interlude on the long journey, equaling the stop at Multnomah Falls on the route of the Northern Pacific Railway at about the same distance from its Portland station, and to a traveler is no more of an objection to the route. It is a delay of fifteen minutes in a journey of 10 000 minutes, which, were it necessary, could be shortened some 3 000 minutes, and yet the trains not be run on what is called "fast time."

Neither a tunnel nor a bridge in this vicinity can be seriously considered for the present, while to remain on the south side of this water is to use part of the route first built by the Central Pacific people, the "Western Pacific Railroad," to a junction at Sacramento with an increased distance of 60 miles, or double the distance by the short "California Pacific," now the main passenger route, our old opposition road, to which feature of directness I recur with pleasure as once its chief engineer.

There is no gainsaying the old axiom, "A straight line is the shortest distance between two points," especially if by traveling it all grades are avoided.

Digitized by Google

Additional Information.

The following additional information in regard to the "Solano" has been obtained by reason of the interest manifested at the presentation of the foregoing paper.

The idea of this large ferry-boat was suggested by Governor Leland Stanford, President of the Central Pacific Railroad Company, and now United States Senator from California.

The design and construction of the "Solano" were entrusted to Mr. Arthur Brown, West Oakland, Cal., Superintendent of the Bridge and Building Department of the Railroad Company, with the only conditions that she should be large enough to convey one of the largest freight trains across the Straits of Carquinez.

The boat generally, the slips, approaches, aprons, machinery, and all other accessories, were designed and built by Mr. Arthur Brown.

The engines were designed and built by the Harlan & Hollingsworth Company, Wilmington, Del.

The boilers were designed and built, and the machinery in the boat was erected, by Mr. Andrew J. Stevens, Sacramento, Cal., the Master of Machinery for the railroad company.

The boat is considered a success in every particular. The hull is of wood; the longest timbers are the deck beams, which are single sticks 115 feet in length. There were a great many sticks of large size up to 100 feet long. The trusses under the tracks take the place of keelsons, so that excessively long sticks were not a necessity for that duty.

The following data are given in regard to the comparative cost and evaporation of fuel, oil and coal:

Coal used in twenty-four hours	18 tons.
Fuel oil used in twenty-four hours	1 620 gallons.
Gallons of oil equivalent to 1 ton of coal	90 "
Indicated horse-power	1 702
Price of coal per ton of 2 000 pounds	$5
Price of fuel oil in May, 1886	4 cts. per gal.
Time of raising steam with coal	1 hour.
" " " oil	40 minutes.
Weight of coal used per cubic foot	$58\frac{6}{10}$ pounds.
" fuel oil " "	$57\frac{15}{100}$ "

There have been no measurements of the actual evaporation accomplished with either coal or oil. As the duty performed by coal and oil is the same, a comparative idea of their efficiency may be obtained from the foregoing data. According to the calculations of those using the

Digitized by Google

boat, the saving by the use of fuel oil is about 18 per cent., principally in labor. A diagram of the mode of using oil has been received.

The force employed to operate the boat is as follows:

ENGINEER DEPARTMENT:

Two watches, each consisting of—
1 Chief engineer.
2 Assistant engineers.
4 Firemen.
1 Oiler.
A total of 16 men in both watches.

OPERATING DEPARTMENT:

Two watches, each consisting of—
1 Captain.
1 First officer.
1 Second officer.
5 Deck hands.
A total of 16 men in both watches.
Total in both departments, 32 men.

The total detentions, including stoppages at both ends of the trip, are about ten minutes.

NOTE.

Since this paper was printed, I have received the following information from Mr. Arthur Brown, Superintendent Bridges & Building Department, Southern Pacific Company, in response to recent inquiry:

"The use of petroleum, as a fuel for making steam on our boats, was discontinued because it was found that, as the boilers were not constructed for such fuel, its further use might result in injury to them. It would seem that for liquid fuel, boilers should be of special construction."

Digitized by Google

The illustrations accompanying the paper on the Railroad Ferry Steamer "Solano" are :

Plate LVI. Ferry Steamer "Solano" in dock.
" LVII. Photograph of Steamer.
" LVIII. Map Showing Straits of Carquinez.
" LIX. Cross-section and Elevation of Boat and Apron.
" LX. Plan of Boat.
" LXI. Plate Showing Details of Construction.
" LXII. Plan of Boat and Steering Gear.
" LXIII. Arrangement for Burning Oil in Boilers.

Digitized by Google

LOUISVILLE CEMENT.

The undersigned is General Agent for the following Standard Brands of Louisville Cement:

FALLS MILLS (J. Hulme Brand),
BLACK DIAMOND MILLS (River),
SPEED MILLS,
FALLS CITY MILLS,
QUEEN CITY MILLS,
ACORN MILLS,
BLACK DIAMOND MILLS (Railroad),
FERN LEAF MILLS,
EAGLE MILLS.

This Cement has been in general use throughout the West and South since 1830, most of the public works having been constructed with it. Orders for shipment to any part of the country, by rail or water, will receive prompt and careful attention.

Sales for 1888, 1,202,118 Barrels.
Sales for 1889, 1,338,464 Barrels.

WESTERN CEMENT ASSOCIATION,

247 W. Main St., Louisville, Ky.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

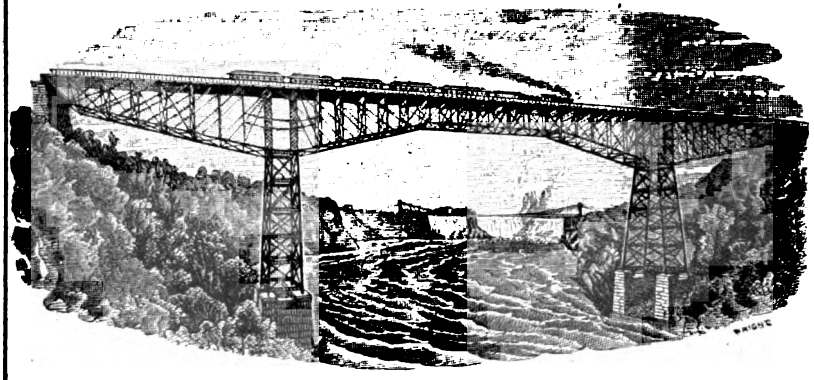
SELECT ADVERTISEMENTS WILL BE RECEIVED AT THE FOLLOWING RATES:

	TWELVE MONTHS.	SIX MONTHS.	THREE MONTHS.	ONE MONTH.
One Page.....	$100.00	$55.00	$30.00	$12.00
One-half Page.....	55.00	30.00	18.00	7.00
One-quarter Page	30.00	18.00	10.00	5.00
One-twelfth "Card".	12.00	.....	.....	....

Address the Secretary of the Society, 127 East 23d Street, New York.

Digitized by Google

UNION BRIDGE CO.



C. MACDONALD, GEO. S. FIELD,

C. S. MAURICE, EDMUND HAYES,

CIVIL ENGINEERS,

And Constructors of Iron and Steel Bridges, Viaducts, Roofs, Elevated Railroads, Marine Piers, etc.

Designs and Estimates will be sent on application to

General Office, WASHINGTON BUILDING,

1 BROADWAY, NEW YORK.

Works: { Athens, Pa. - - Capacity, 15,000 Tons.
Buffalo, N. Y. - - " 15,000 "

Digitized by Google

VII 570

MAY.

1890.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Library.

CONTENTS.

TRANSACTIONS. VOL. XXII.

No. Page

437. A New Graphical Solution of the Problem, What Position a Train of Concentrated Loads must have in order to cause the Greatest Stress in any given part of a Bridge Truss or Girder. HENRY T. EDDY..... 259

ILLUSTRATIONS.

Twenty-six Figures, Illustrating Paper No. 437.

The Transactions of the Society will be sent to any subscriber at the rate of $10 per year; and to clubs of ten or more, when ordered through the Secretary of an Engineering or Technical Society or Club who will be responsible for the payment, at 25 per cent. discount.

PUBLISHED AT THE HOUSE OF THE SOCIETY, 127 EAST TWENTY-THIRD STREET, NEW YORK, ON THE SECOND WEDNESDAY OF THE MONTH.

Entered according to Act of Congress, in the year 1890, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Digitized by Google

American Society of Civil Engineers.

OFFICERS FOR 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN, WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Standing Committees.

On FINANCE :—Charles B. Brush, Mendes Cohen, Robert Van Buren.
On LIBRARY :—Alphonse Fteley, Theodore Voorhees, William Ludlow.

Special Committees.

THE PRESIDENT OF THE SOCIETY IS HONORARY MEMBER OF ALL COMMITTEES.

ON UNIFORM STANDARD TIME :—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston, Fred. Brooks.

ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY :—F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, A. V. Abbott.

ON UNIFORM METHODS OF TESTING MATERIALS USED IN METALLIC STRUCTURES, AND ON REQUIREMENTS FOR THESE MATERIALS TO FURTHER IMPROVE THE GRADE OF SUCH STRUCTURES:—G. Bouscaren, William H. Burr, J. G. Dagron, R. W. Hunt, H. G. Morse, P. Roberts, Jr., H. B. Seaman.

MEETINGS.

Regular Meetings, on the first and third Wednesdays of the month, at 20 o'clock, except during July and August.

The rooms of the Society are open from 9 to 17 o'clock each business day, except Saturday, when they are closed at 15 o'clock.

The Library and Conversation Room will also, for the present, be open every *Wednesday Evening*, from 19½ to 22 o'clock. Members are invited to avail themselves of the opportunities afforded on these evenings, both for consultation of books and periodicals, and for conversation.

HOUSE OF THE SOCIETY—127 EAST TWENTY-THIRD STREET, NEW YORK.

Digitized by Google

SEP 10 1890

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

437.
(Vol. XXII.—May, 1890.)

A NEW GRAPHICAL SOLUTION OF THE PROBLEM, WHAT POSITION A TRAIN OF CONCENTRATED LOADS MUST HAVE IN ORDER TO CAUSE THE GREATEST STRESS IN ANY GIVEN PART OF A BRIDGE TRUSS OR GIRDER.

By HENRY T. EDDY, C.E., Ph.D.

INTRODUCTORY.

1. The object of this paper is to introduce a new graphical method for determining what position a moving train of wheel weights must have in order to produce the greatest stress in any given part of the bridge truss or girder over which the train is passing.

The method which will be here proposed depends principally upon the construction and use of a class of polygons or curves, which we have named reaction polygons. One of these is the line of shears, otherwise called the load line, which has been long used to represent to the eye the distribution of shears or loads in the span; but its graphical possibilities seem to have been little studied. One valuable proposition respecting the load line has, however, been in constant use here since 1875, by Mr. C. L. Strobel, M. Am. Soc. C. E., and his successors in the office of the Cincinnati Southern Railway, who learned it in 1871 as a student of

Digitized by Google

Professor Mohr, then at Stuttgart, now at Dresden. This paper will be mostly devoted to the development of numerous propositions of a character similar to that one.

Reaction polygons, as will appear, are readily constructed graphically, and their properties are such as to give with ease the train positions for maximum stresses as well as to decide which of several maxima is the greatest. The proof of these constructions is given in algebraic form, or, rather, the graphical constructions are merely representations of the algebraic conditions for maximum stresses. A number of the propositions of this algebraic theory have been long known to the profession,* without mentioning the single case of maximum bending moment before mentioned as having been worked out algebraically and graphically by Professor Mohr before 1871.

The author has enlarged the algebraic theory by applying it to many cases not before treated, especially in cantilevers and double intersection trusses, but, as before stated, the principal object has been to show how the algebraic theory leads to convenient graphical solutions of the equations of condition for maximum stresses.

A general description of this method was given by the author in a paper entitled "A Contribution to the Theory of Maximum Stresses in Bridges Under Concentrated Loads," read before the American Association for the Advancement of Science," at the Buffalo meeting, August, 1886. A paper entitled "Reaction Polygons and their Properties" was also read by him before the same association at their New York meeting, August, 1887. Both these papers were printed by abstract only.

Part I treats the simple girder and single intersection truss resting upon end supports. It contains a full explanation of the properties and methods of construction of reaction polygons and the investigation of all problems of maximum stresses in girders and single intersection trusses by their aid.

Part II treats cantilevers with two supports and double intersection trusses with end supports. It shows how the same simple graphical methods suffice to resolve also all important questions of maximum stress, under concentrated loads, for bridges of these more complicated forms.

---

* William Cain, "*Van Nostrand's Magazine*," March, 1885; G. F. Swain, *Transactions* Am. Soc. C. E., July, 1887; W. H. Burr, Bridge and Roof Trusses, 3d Ed., 1886; Ward Baldwin, *Engineering News*, September 28th, October 12th, and December 28th, 1889.

Digitized by Google

PART I.

Maximum Stresses in Simple Girders and Single Intersection Trusses with End Supports, under Concentrated Loads.

2. Let the stepped line $k_1\ h_1\ k_2 \ldots . h_4$ of Fig. 1 represent the relative magnitude and position of a system of wheel-weights, $W_1 \ldots . W_4$, situated at invariable distances from each other. Call this stepped line the *load line* or *weight polygon*.

Suppose this train of wheel-weights to move across a girder of span, $l = A\ B$.

Lay off $k_1\ f_1$, $k_2\ f_2$, $k_3\ f_3$, and $k_4\ f_4$, respectively equal to l, as also $h_1\ g_1$, $h_2\ g_2$, $h_3\ g_3$, and $h_4\ g_4$. Join each of these points f and g to its corresponding k and h respectively, as shown in Fig. 1.

Now, if a girder of span, l, and supported at its ends, occupy the position $A\ B$, then that part of the vertical at A which lies between the lines $f_2\ k_2$ and $f_2\ h_2$ represents so much of W_2 as is supported at A, and that part of the same vertical which lies between $f_3\ k_3$ and $f_3\ h_3$ represents so much of W_3 as is supported at A, and similarly for W_4.

The sum of these quantities is the total load resting upon A, and is equal to the reaction of the pier at A. Let $h_a\ u_a = V'$ denote this reaction at A. Then as the train is moved across the span from right to left (or what is the same thing, as the span is shifted toward the right), let the reaction V' be taken at each point, and let it be laid off upwards from the weight line. The upper extremity u_a of V' will describe some polygon $u_1\ u_2$. The broken line or polygon $u\ u$ described by u_a as the span $A\ B$ is thus shifted, we shall define as the *reaction polygon* of A for the span $A\ B$. This polygon, which is the locus, or path described by u_a is readily constructed as follows: Since the only reaction when A is situated between the verticals at f_1 and f_2 is that due to W_1, it is evident that the vertical distance between $f_1\ k_1$ and $f_1\ h_1$ is the reaction at A; hence $f_1\ f_2'$ is one side of the reaction polygon. The reaction at any point between the verticals at f_2 and f_3 is the sum of that due to the weights W_1 and W_2; hence f_2' and f_3' is a side of the reaction polygon when f_3' be so placed in the vertical through f_3 that the height of f_3', above $f_1\ k_1$, is the sum of the segments of this vertical included within the triangles $f_1\ k_1\ h_1$ and $f_2\ k_2\ h_2$. The polygon extends thence to u_1, so situated in the vertical at W_1 that $h_1\ u_1$ is equal to the sum of the segments of this vertical which are included within the triangles $f_2\ k_2\ h_2$, and $f_3\ k_3\ h_3$, etc., etc. There will in general be an angle of this polygon in each vertical in which there is a weight and in each vertical at a distance l to the left of each weight. At each such vertical it will be necessary to take the sum of the segments of this vertical which li

Digitized by Google

within all the triangles of the type $f\,k\,h$ which the vertical cuts, a process of summation not difficult to effect graphically with compasses.

The reaction polygon at B, the other extremity of the span $A\,B$, can be found from the segments of verticals included within the triangles of the type $g\,h\,k$ by a similar process of summation. It is, however, quite unnecessary to actually perform this second summation, for, if we denote the reaction at B by $V'' = h_b\,v_b$, it is evident that $V' + V''$ is the total load upon the span $A\,B$. Hence $u_a\,v_b$ must be horizontal, *i. e.*, v_b is situated on the horizontal through u_a at a distance $l = A\,B$ towards the right. The reaction polygon for the point of support B then differs from that for A only in being displaced a distance l to the right of that for A. As they differ only in horizontal position, they may be said to be parallel. The horizontal line $u_a\,v_b$ may be called the line of *zero shear* for the span $A\,B$. The fitness of this name is evident when we notice that the shear at any assumed point of the span $A\,B$ is measured by the vertical distance at that point from the line of zero shear to the stepped weight line $h\,k$. The maximum shear, positive or negative, at any point C of the span $A\,B$, may be readily found by trial thus:

Cut a rectangular card of width $A\,C$, and call its upper edge $A'\,C'$. Now, keeping $A'C'$ horizontal, slide A' along the reaction polygon $u\,u$. The upper edge $A'C'$ may be regarded as part of a movable line of zero shear. Find by trial the position at which C' is farthest above (or below) the weight line. This will give the position of $A\,C$ for the maximum (or minimum) shear at C. Such trial is simplified by reason of the fact that the maximum shear occurs just in front of one of the forward wheels, near the head of the train, a proposition which will be established later on. It is unnecessary to determine the minimum shear, as it will be surpassed in magnitude by the maximum which occurs near the head of the train in passing over the span in the opposite direction, which is consequently the true minimum. The question of maximum shear will, however, be considered in a more general manner and more at length hereafter.

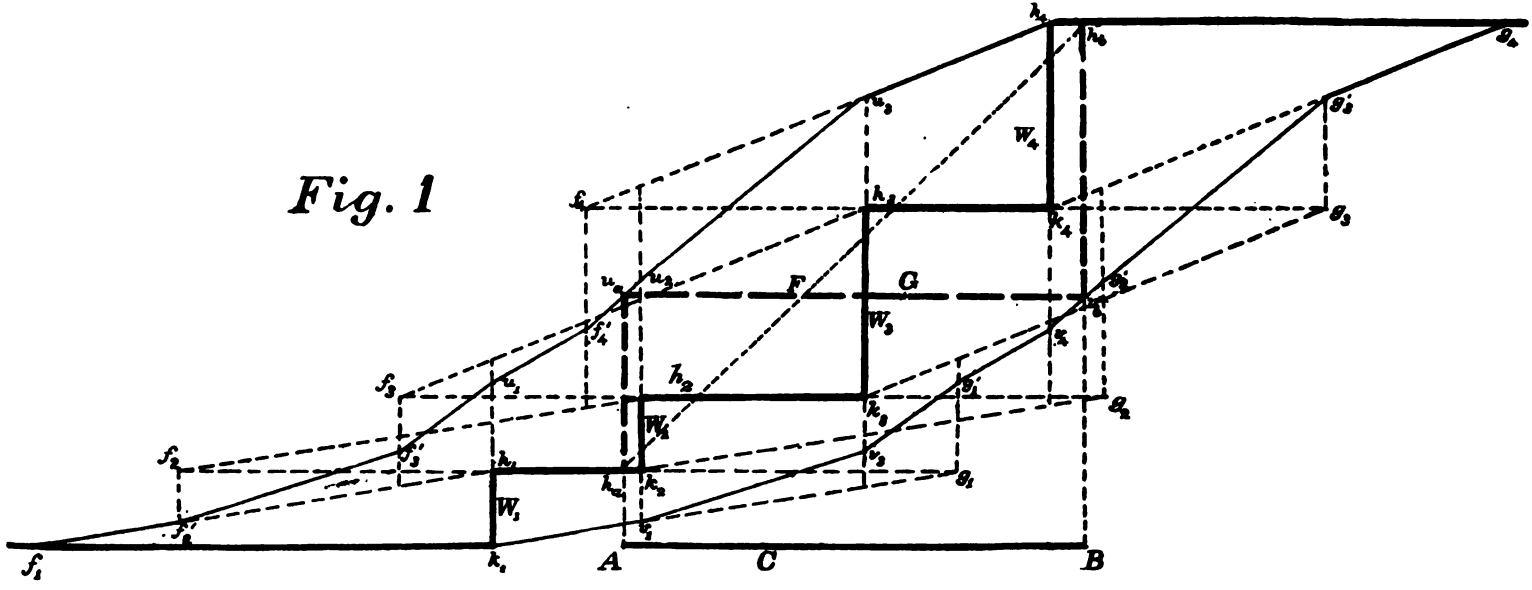
The maximum value of V' as the train passes over the girder can be readily determined by inspection, as it is the greatest vertical distance from the reaction polygon $u\,u$ to the weight line $h\,k$; this is also the maximum shear at and near the extremity A of the span $A\,B$.

The position of the center of gravity of the total load upon the span $A\,B$ may be readily constructed, as follows:

Draw $h_a\,h_b$ cutting $u_a\,v_b$ the line of no shear at F, then this point F divides the span $u_a\,v_b$ proportionally to the reactions V' and V''. Now lay off $u_a\,G = v_b\,F$, then the point G divides the span inversely as the reactions V' and V'', consequently the center of gravity of the total load upon the span $A\,B$ is in the vertical through G.

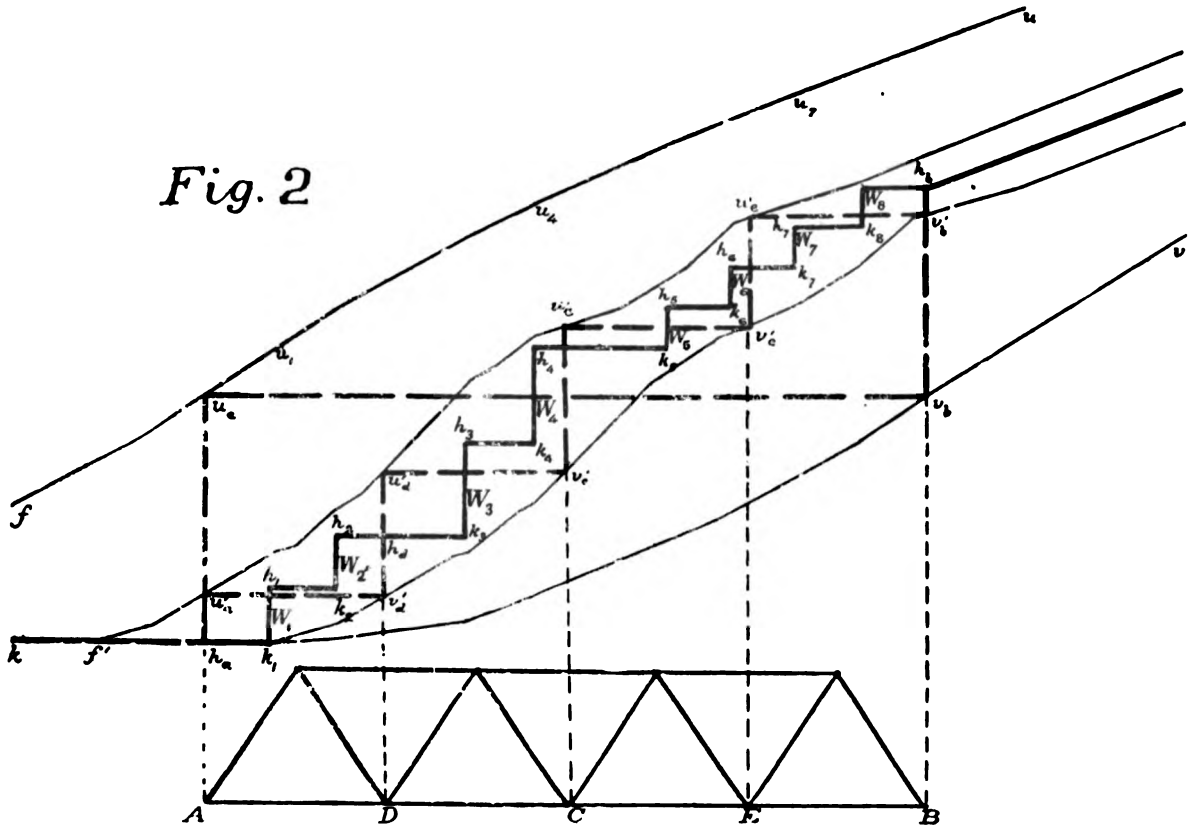
Digitized by Google

Fig. 1



Digitized by Google

Fig. 2



Digitized by Google

3. In any single intersection truss the total load carried by the stringers to any pin point, as well as the shear in the stringers of any panel, can be found for any position of a given train by help of reaction polygons, as follows:

Let the stepped weight $h\,k$ of Fig. 2 represent the given train, which may be taken to consist of a locomotive and tender followed by a uniform unlimited train.

Suppose this train to stand upon a single intersection truss of length $l = A\,B$, whose panels are each of length $p = A\,D$. Draw the reaction polygons $u\,u$, $v\,v$, for the span l, and $u'\,u'$, $v'\,v'$, for the span p. Now if the truss be taken in the position $A\,B$, the total reaction at A due to the load upon the span $A\,B$ will be $h_a\,u_a$, and similarly the reaction at A due to the load upon the panel $A\,D$ will be $h_a\,u_a'$.

Likewise, $h_b\,v_b$ is the reaction at B of the total load upon $A\,B$, and $h_b\,v_b'$ that of the load upon the panel $E\,B$. Hence $u_a\,u_a'$ is the shear in the web members of the panel $A\,D$. Again, $h_d\,v_d'$ is the reaction at the pin point D due to the wheels upon $A\,D$, and $h_d\,u_d'$ is that due to the wheels upon $D\,C$.

Hence, $v_d'\,u_d'$ is the load transmitted by the stringers in the panels $A\,D$ and $D\,C$ to the pin at D. Similar statements hold respecting the loads supported at the pin points C and E, viz.: $v_c'\,u_c'$ is the pin load at C, and $u_e'\,v_e'$ that at E. Also the shear in the web members of any panel is the vertical distance between the line of zero shear $u_a\,v_b$ for the span $A\,B$ and the line of zero shear for the panel considered, *i. e.*, $u_a'\,v_d'$ is the line of zero shear in $A\,D$, $u_d'\,v_c'$ that in $D\,C$, etc. The stepped line $u'\,v_d'\,u_d'\,v_c'\ldots$. is the line of zero shears for the panels. Furthermore, it will be easy to find by inspection what position during the passage of the train produces the greatest shear in the end panel $A\,D$, by observing at what point the vertical distance $u_a'\,u_a$ between the reaction polygons $u\,u$ and $u'\,u'$ is greatest. What the position must be to produce the greatest shear in any panel other than the end panel will be considered later. The shear in the stringer of any panel is the vertical distance between the line of zero shear for that panel and the weight line, *e. g.*, in $D\,C$, the shear borne by the stringer is represented by the vertical distance between $u_d'\,v_c'$ and the weight line.

4. The greatest load at a pin point is evidently the greatest vertical distance between the polygons $u'\,u'$ and $v'\,v'$. The position where the pin point experiences its greatest load can easily be found by inspection. In Fig. 2 it is at W_3; hence, during the passage of the train across the truss $A\,B$, whenever the wheel W_3 reaches either of the pin points D, C or E, the load at that pin is then greater than for any other position of the train. But, in order to facilitate the investigation it will be useful to obtain the analytical condition for a maximum pin load as follows: Let the distance from E to any wheel weight be denoted by e and its distance from D by d, both these co-ordinates being reckoned as positive toward the left from the origins E and D.

Digitized by Google

Then the total pin load R at C may be written—

$$R = \frac{1}{p}\left(\sum_{E}^{C} W e - \sum_{C}^{D} W d \right)$$

in which equation p is the length of either of the equal panels $E\,C$ and $D\,C$, and d is negative for each of the wheels in the panel $D\,C$.

Now, let the train move a small distance $\delta\,x$ toward the left, which movement will change e to $e + \delta\,x$ and d to $d + \delta\,x$, and R to $R + \delta\,R$.

Then we obtain in the new position

$$R + \delta\,R = \frac{1}{p}\left[\sum_{E}^{C} W (e + \delta\,x) - \sum_{C}^{D} W (d + \delta\,x) \right]$$

Hence, by subtraction and division—

$$\frac{\delta\,R}{\delta\,x} = \frac{1}{p}\left(\sum_{C}^{E} W - \sum_{C}^{D} W \right) = \frac{1}{p}\,(P_e - P_d) \ldots\ldots\ldots\ldots (1)$$

in which equation P_d is the load in the panel $D\,C$, and P_e that in $C\,E$. In order that R may reach a maximum value as the train moves to the left, the quantity $P_e - P_d$ must change from positive to negative. This change can only occur in one way, viz.: Some wheel reaches C which is of such magnitude that in passing from P_e at the right to P_d at the left of C, the last member of equation (1) changes from + to —. The change can occur in this way only, for when a wheel enters $E\,C$ from beyond E, the effect of this is to increase P_e, and when a wheel leaves $D\,C$ at D the effect is to decrease P_d. Either of these occurrences might change the last member of equation (1) from — to +, but not from + to —; and hence, either might cause the value of R to reach a minimum, but not a maximum value.

It has therefore been shown that the only verticals where a maximum value of R can be found are those in which the wheel weights are found, and this restriction will materially facilitate the inspection by which to discover the position of the train when the value of R is a maximum. The maximum value of R might also be shown to be at a wheel when the panels are of unequal length.

5. A system of reaction polygons consists of a number of such polygons belonging to a given train of wheel weights, in which system its several members are constructed to give the reactions for different lengths of span.

The large diagram, Fig. 3, contains several members of such a system due to the train of wheel weights represented by the weight line there shown. The successive reaction polygons are constructed for spans of 5, 10, 15, 20, 25 and 50 feet respectively.

Digitized by Google

The assumed load consists of two consolidation locomotives and tenders, preceded and followed by a uniform train. The scale of weights is given upon the diagram in pounds, and the scale of distances in feet. It will, however, be convenient to have the scale of distances subdivided more than in Fig. 3; it should be divided to single feet or even less than that.

In the solution of the various problems which follow, we shall presuppose that such a system of reaction polygons has been constructed. In practice it will be necessary to draw them upon a sheet permanently mounted, and to a scale sufficiently large to admit of convenient comparisons. For long spans the diagram must include more of the uniform load than is found in Fig. 3. The successive polygons must be constructed for spans which differ so little in length that it shall be possible to interpolate for spans of intermediate length with approximate correctness. It will appear, however, that the successive polygons need not be very near together in order to solve with entire correctness most of the problems proposed. This arises from the circumstance that most of these problems have one peculiarity in common, a peculiarity which was also exemplified in the problem just discussed as to the maximum value of the pin load R. This becomes evident when we notice that the diagram is not employed to determine at this time the actual magnitude of the maximum value of R; its use merely serves to point out which wheel should stand at the pin point under consideration in order that R may reach a maximum at that pin.

The construction is then employed in the first instance in this and other problems in enabling us to select with certainty which one of the wheels produces the maximum. Although in the problem of the maximum load upon a pin it is also feasible to measure with considerable certainty the magnitude of the maximum value of R, in many of the problems to be treated, this will not be possible. Especially in problems pertaining to the maximum value of the bending moments, it will not be possible to learn anything respecting the absolute value of these moments from the reaction polygons, which will be used merely to discover the position of the train producing a maximum. It is evident that no very close approximation will be needed in most cases to distinguish between different wheels and to discover which one fulfills the conditions demanded of a maximum, for the process consists merely in a selection of one from several quantities which differ by finite amounts. For example, it could in general be determined with certainty and ease from Fig. 3 which of the wheels must stand at a pin point to make R a maximum, not only when the panel length is either 5, 10, 15, 20 or 25 feet, but also when it has any intermediate value.

It will be noticed that the reaction polygons approach in character to regular curves as the spans become longer, and that the successive members also become more and more nearly parallel to each other.

Digitized by Google

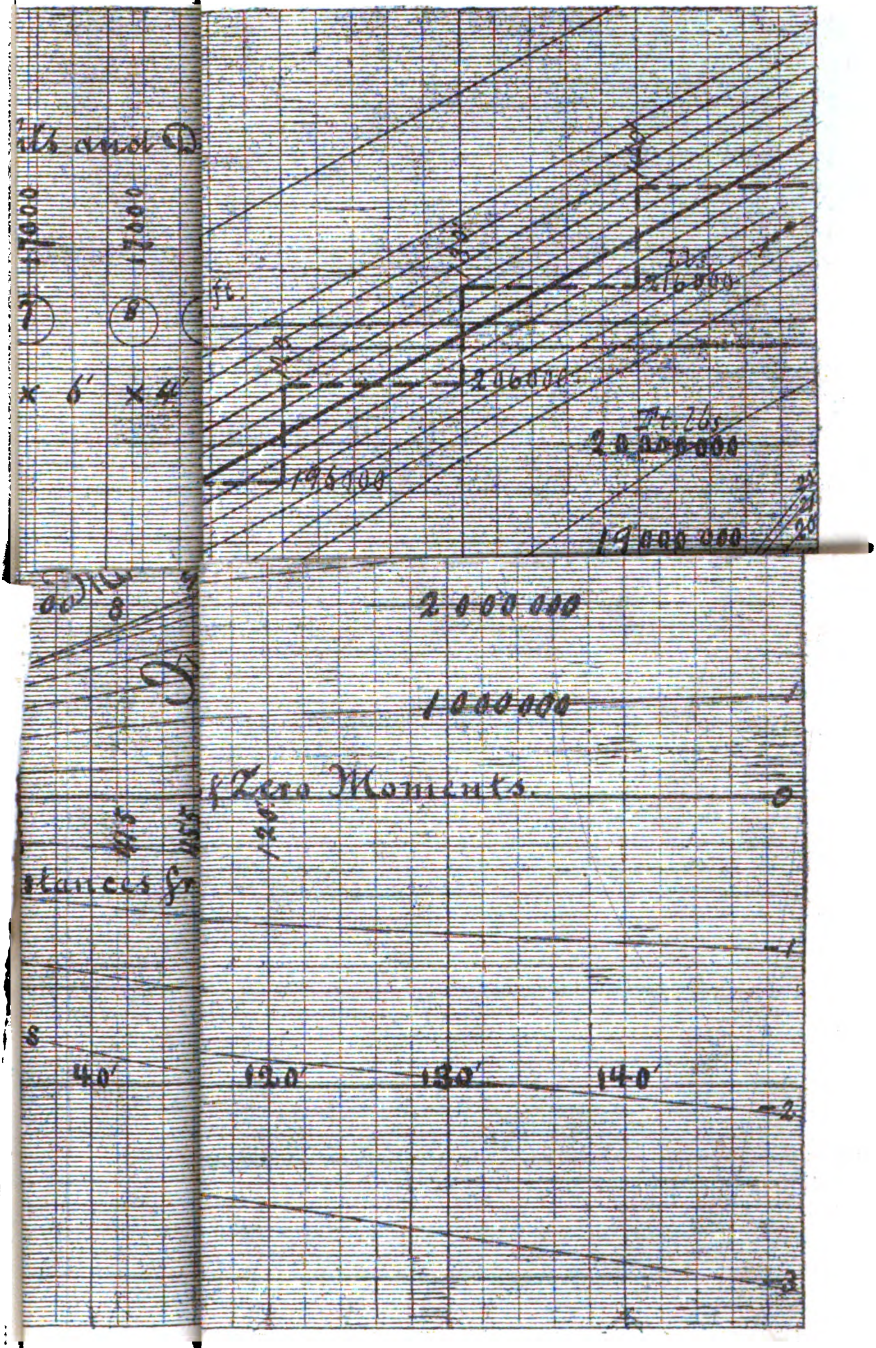
From this it appears that the accuracy of interpolation is greater for long spans than for short ones, provided the successive polygons are those for spans which increase successively by equal increments of length. Now, the labor of constructing reaction polygons increases as the span increases. It is advisable, then, in constructing a permanent sheet, similar to Fig. 3, for practical use, to make the polygons near together for short spans and further apart for longer spans.

It may be noticed that the weight line is itself is the reaction polygon for a span whose length is zero, and hence the discussion of the properties of the weight line constitutes an important part of the subject under consideration. In particular, many of the principal problems respecting maximum stresses in the various members of the single intersection truss are solved by the weight line alone. It is further to be noticed that this method will find its field of greatest usefulness where a number of girders or bridges are to be computed for the same specified train of wheel weights, for then a single sheet, containing a system of reaction polygons due to that train, will suffice for any number of determinations. The permanent sheet of reaction polygons will be much more useful if drawn upon cross section or profile paper, as the ruling will very essentially facilitate its use.

Moreover it will be quite unnecessary to put upon the permanent sheet more than one of the pair of like reaction polygons for each length of span, say, those at the pier A, all of which lie above the weight line. If, for example, it should then be required to find the greatest value of R, it would merely be necessary to use a rectangular card upon which the panel length $A\,D$, Fig. 2, is laid off on the lower edge. Call the lower right-hand corner of the card v_d', and lay off $u_a'\,v_d' = A\,D$. To use the card, slide it on the permanent sheet in such a manner that while the lower edge remains horizontal the point u_a' moves along the reaction polygon, due to the span $A\,D$. The point u_d' will then be found where the right edge cuts $u'\,u'$. Inspection will readily show at which wheel-weight the right-hand edge must be placed in order that the magnitude of $R = v_d'\,u_d'$ may be greatest.

6. The maximum shear at any point, C, of a simple girder, whose span is $A\,B = l$, is known to occur under one of the wheels near the head of the train. To find the greatest shear at any point, it will only be necessary to effect a comparison between the shears under several of the wheels near the head of the train. By help of the reaction polygons the shear under any wheel may, in general, be readily compared with that under any other wheel. To take this more general case first, let the points, $A\,C\,B$, of the girder occupy initially same position, $A'\,C'\,B'$, as represented in Fig. 4, and let them then be shifted toward the right to a second position, $A''\,C''\,B''$, so situated with respect to the train, which is regarded as stationary, that $A'\,A'' = x$.

Digitized by Google

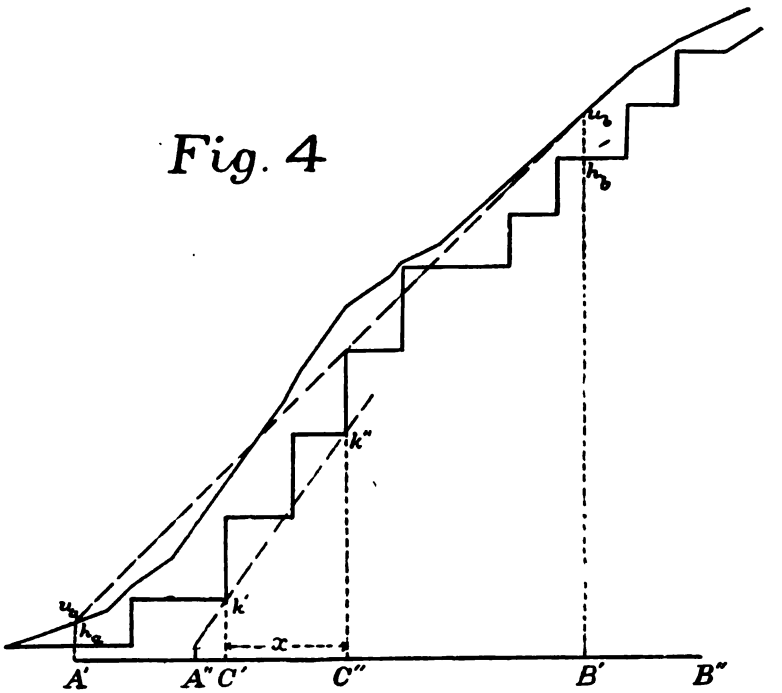


Digitized by Google

Digitized by Google

This is equivalent to moving the train upon the girder a distance x toward the left.

Fig. 4



Let S'' denote shear at C'', and S' that at C', then:

$$S'' = \frac{1}{l} \sum_{A''}^{B''} W b'' - \sum_{A''}^{C''} W$$

$$S' = \frac{1}{l} \sum_{A'}^{B'} W b' - \sum_{A'}^{C'} W$$

in which equations W denotes any wheel-weight between the limits indicated, b'' and b' the distances from B'' and B', respectively, to any such wheel, taken as positive toward the left.

By subtraction we find:

$$S'' - S' = \sum_{A''}^{B''} W \frac{b''}{l} - \sum_{A}^{B'} W \frac{b'}{l} - \sum_{A''}^{C''} W + \sum_{A'}^{C'} W$$

But, $A'' B'' = A' B' - A' A'' + B' B''$
and, $A'' C'' = A' C' - A' A'' + C C''$

Digitized by Google

Now, change the limits in accordance with these last equations.

$$\therefore \quad S''-S' = \left[\sum_{A'}^{B'} - \sum_{A'}^{A''} + \sum_{B'}^{B''}\right] W\frac{b''}{l} - \sum_{A'}^{B'} W\frac{b'}{l}$$

$$- \left[\sum_{A'}^{C'} - \sum_{A'}^{A''} + \sum_{C'}^{C''}\right] W + \sum_{A'}^{C'} W,$$

as may be readily seen by noticing the segments over which the compound summations extend. Now, evidently, $b'' - b' = x$, and if we place $b'' = a'' + l$, then is a'' the distance from A'' to any wheel, taken as positive toward the left. By the help of these relations the foregoing equation may be reduced to the form

$$\frac{S''-S'}{x} = \frac{1}{l}\left(L' - \sum_{A'}^{A''} W\frac{a''}{x} + \sum_{B'}^{B''} W\frac{b''}{x}\right) - \frac{X}{x} \ldots\ldots\ldots (2)$$

in which equation L' is employed for brevity to denote the total initial load, *i. e.*, the sum of the wheel weights between A' and B', while X denotes the sum of the wheel weights between C' and C''.

It will be noticed that, since all the wheels between A' and A'' are at the left of A'', the value of a'' for each of these wheels is essentially positive.

The last three terms of equation (2) take account respectively of the effect of any wheels which during the motion pass off from the girder $A\,B$ at A, of any which enter upon it at B, and of any which pass C, when the initial and final positions differ by any amount x.

If S' be taken to be the shear at C' just at the left of any one wheel, and S'' that at C'' just at the left of any other wheel, then equation (2) will show which wheel produces the greater shear at C.

In Fig. 4 the vertical height of h_b above h_a is L', the load between A' and B',

also $$\sum_{A'}^{A''} W\frac{a''}{x} = \text{the ordinate } h_a\,u_a,$$

and $$\sum_{B'}^{B''} W\frac{b''}{x} = \text{the ordinate } h_b\,u_b,$$

provided u_a and u_b are situated upon the reaction polygon for a span of length x; for these are the expressions for the reactions at A' and B', the left extremities of the spans $A'\,A''$ and $B'\,B''$. Hence

$$\frac{1}{l}\left(L' - \sum_{A'}^{A''} W\frac{a''}{x} + \sum_{B'}^{B''} W\frac{b''}{x}\right)$$

Digitized by Google

is the tangent of the angle which the sloping line $u_a\ u_b$ makes with the horizontal line $A'\ B'$, and it therefore may be taken as the measure of its inclination or slope.

Again, X is the total load between C' and C'' and $X \div x$ is the tangent of the angle which $k'\ k''$ makes with $A'\ B'$. If the slope of the line $u_a\ u_b$ is steeper than that of $k'\ k''$ then $S'' > S'$, but if $k'\ k''$ is the steeper then $S'' < S'$.

It is to be noticed that any slight vertical errors in the positions of u_a and u_b, such as might arise in obtaining these points by interpolation in the manner previously mentioned, will have little effect in causing error in the slope of $u_a\ u_b$.

This comparison of the relative slopes of $u_a\ u_b$ and $k'\ k''$ will not require the permanent sheet containing the system of reaction polygons to be defaced by drawing these lines upon it, for by using two rulers or a ruler and stretched thread the comparison can be readily effected.

Equation (2) can be still further utilized to determine the segments of the span in which the maximum shear is dominated by the successive wheels near the head of the train, in case the train is taken to consist of one or more locomotives followed, but not preceded by a uniform train. Such a disposition of load is known to cause the severest shear.

It is sufficiently evident that as the train of wheel weights represented in Fig. 5 enters upon the span at B, the shear for same distance near B is greater under W_1 than under any wheel following it.

In order to find the point where the shear just in front of W_2 is first equal to the shear just in front of W_1, let S_2 and S_1 denote the shears just in front of W_2 and W_1 respectively; then in equation (2) we have $S_2 = S_1$ and $X = W_1$, while x is the distance from W_2 to W_1. Equation (2) may then be put in the form

$$L' + \sum_{B'}^{B''} W \frac{b''}{x} = W_1 \frac{l}{x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

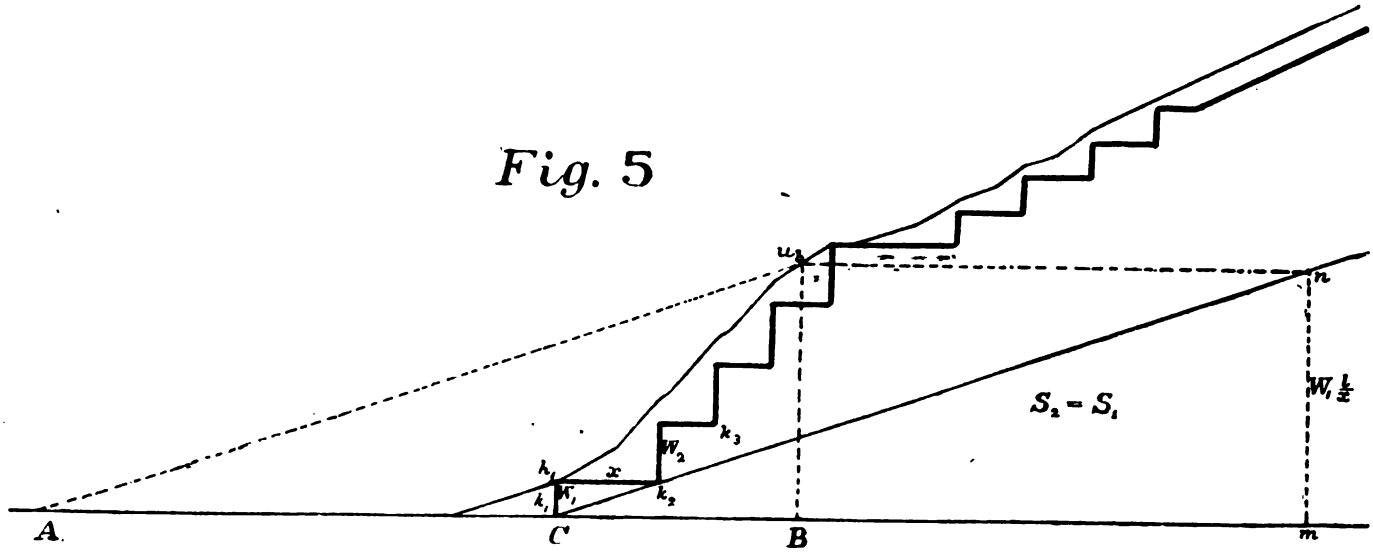
for the term

$$\sum_{A'}^{A''} W \frac{a''}{l}$$

vanishes because no part of the train is supposed to have reached A'' by the time $S_2 = S_1$. Prolong $k_1\ k_2$, which has the slope $W_1 \div x$ until it cuts the vertical $m\ n$ at a distance from k_1 such that $k_1\ m = l = A\ B$, then is $m\ n = W_1\ l \div x$. Through n draw a horizontal line cutting the reaction polygon for the span x at u_b. Then, if $A\ B = l$, the point k_1 is the point C of the span $A\ B$ where S_2 first becomes equal to S_1, and the maximum shear in the segment $B\ C$ is dominated by the wheel W_1, for the train is then in a position upon $A\ B$, which satisfies equation (2). To the left of C is the segment dominated by W_2.

Digitized by Google

Fig. 5

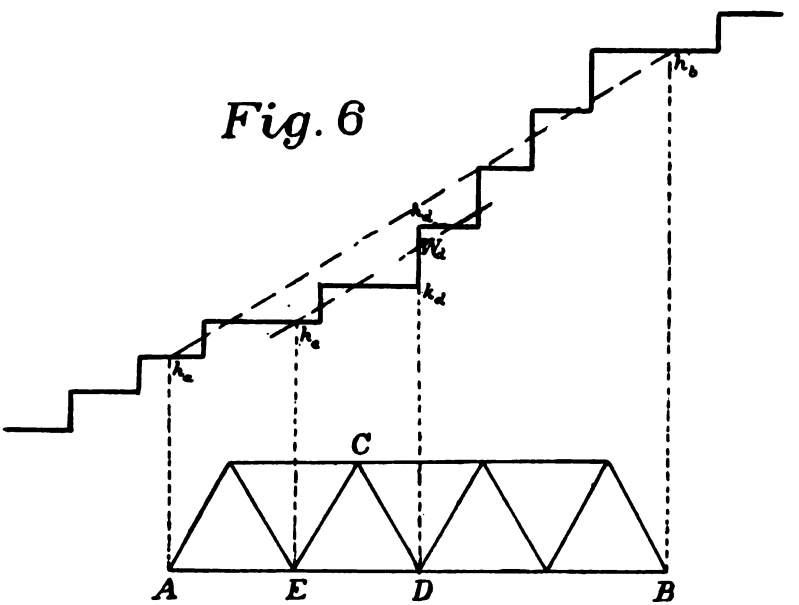


Digitized by Google

It should be noticed that if the slope line $S_2 = S_1$ is drawn upon the permanent sheet, then the point B, and so the segment $B\ C$ dominated by W_1, is readily found for any span l without drawing any other lines upon the sheet. Similarly it will be possible to find where S_3, the shear under W_3, first becomes equal to S_2, in case this occurs before W_1 leaves the span; and the distance between the point so found and the point C, found above, will be the segment dominated by W_2.

But there are seldom more than two or three such segments, for the shear is dominated in any ordinary case by one of the wheels very near the head of the train. The greatest shear near A, after one or more wheels has left the span, has been previously discussed.

Fig. 6



It is also to be noticed that when w_b, and so C, is not very accurately determined, this is a matter of small consequence, for the shear under W_1 is very nearly equal to that under W_2 for all points near C, when thus determined. However, it will be well to have the reaction polygon upon the permanent sheet for a span equal to the exact distance between W_1 and W_2.

7. The position of the train for the maximum shear in the web members of any panel $D\ E$ of a single intersection truss $A\ B$ with parallel chords can be found from the weight polygon alone, as may be shown in the following manner:

Digitized by Google

In Fig. 6 let S denote the shear in the web members of any panel D E, then is S the difference between the shear at C in the span A B, and the shear at C in the span D E, as was shown in connection with Fig. 2. Hence

$$S=\sum_{A}^{B} W\frac{b}{l}-\sum_{A}^{C} W-\sum_{E}^{D} W\frac{d}{p}+\sum_{E}^{C} W,$$

in which equation d is the distance from D to any wheel in the panel D E of length p, and b is the distance from B to any wheel in the span A B of length l.

Now let the train move some small distance δx toward the left, which will change S, b and d respectively into $S+\delta S$, $b+\delta x$ and $d+\delta x$, and we then obtain by subtraction a result which may be written in the form

$$\frac{\delta S}{\delta x}=\frac{L}{l}-\frac{P}{p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which equation L denotes the total load upon A B, and P that upon D E. In order that S may reach a maximum value, the last member of equation (4) must change from positive to negative, which can only occur by one of two ways as the train moves toward the left, viz.: either by the entrance of a wheel upon D E at D, or by the departure of a wheel from A B at A; for the former increases P and the latter decreases L. S may reach a maximum value from either of these causes several times during the passage of a train, but is more likely to do so when a wheel enters upon D E at D than when a wheel leaves A B at A, because in equation (4) P has only the small denominator p, whereas L has the denominator l, which is several times larger than p. Hence any change in the value of P exerts a more decisive influence upon the sign of the last member of (4) than the same change in L does.

Whenever a wheel weight W_d arrives at D, as shown in Fig. 6, such that the slope of the line $h_e k_d$ is less, but the slope of the line $h_e h_d$ is greater than the slope of the line $h_a h_b$, it is evident that the last member of equation (4) will be positive or negative, according as W is regarded as not standing upon the panel D E, or as standing upon it, and the sign of (4) will change from positive to negative, as W_d enters the panel D E.

Whether W_d causes a maximum shear in entering D E, may be discovered most easily by laying down a parallel ruler in the position h_a h_b, and then finding whether the parallel to it through h_e cuts W_d or not. If it does cut W_d, then the last member of (4) changes from + to — as W_d enters upon D E at D, and S is at a maximum for that wheel position.

The work will be facilitated by marking the points A E D B upon the edge of a card and sliding the card upon the permanent sheet to

Digitized by Google

positions such that D is successively made to lie in the verticals of the successive wheels.

A still more convenient method for discovering the wheel position for a maximum value of S in the end panel has been previously explained in connection with Fig. 2.

Again, when any wheel weight (as $W_a = k_a\, h_a$ say) leaves the span $A\ B$ at A, there will be two slope lines $h_a\ h_b$ and $k_a\ h_b$ for the span $A\ B$. In case one of these has a greater and the other a less slope than that of the panel slope line $h_e\ h_d$, then S will reach a maximum value by the departure of W_a from the span $A\ B$ at A. Practically, whenever a parallel to $h_e\ h_d$ passed through h_b cuts any wheel weight W_a, then S is a maximum in $D\ E$ with W_a standing at A.

In like manner it may be shown that the shear in $D\ E$ reaches a minimum value either by the departure of some wheel from $D\ E$ at E, or by the entrance of some wheel upon $A\ B$ at B, and more probably in the former case than in the latter.

That part of the graphical process just given which relates to finding the greatest shears in simple trusses was worked out by the author in 1885, and communicated to Ward Baldwin, M. Am. Soc. C. E., who published it in the *Engineering News*, September 28th, 1889, together with some of the other processes used by him in practical work, he giving the author due credit in *Engineering News*, December 28th, 1889, upon learning that the author had not yet published the process.

Now, it is well known that that one of the several maxima values of S which may occur at any point of the span during the passage of the train is in general the greatest which is nearest the head of the train. It will then be most convenient to find those segments of the span in which the maximum value of S is dominated by the successive wheels at the head of the train, which segments may be successively determined as follows: First find where the last member of equation (4) vanishes in case $P = W_1$, *i. e.*, in case $\frac{W_1}{p}\, l = L$; which last equation serves to determine the value of L in case $P = W_1$, and serves also to determine the position of the train and the length of segment in which the maximum value of S is dominated by W_1. For, in Fig. 7, let $k_1\ i = p$ be the panel length; and let $i\ i_1 = W_1$. Draw the line $k_1\ i_1$ and prolong it to j_1. Lay off $k_1\ m = l$ the length of the span, then by similarity of triangles we have $m\ j_1 = W_1\ \frac{l}{p}$. Draw a horizontal line through j_1: it cuts W_2; and it is evident that so long as W_1, or W_1 and W_2, stand upon the span, but W_3 does not, the last member of equation (4) will be negative or positive, according as W_1 enters the panel under consideration or not, *i. e.*, the maximum value of S is dominated by W_1 in all those panels of the span of which the right-hand panel points are reached by W_1, provided at least one wheel but not more than two

Digitized by Google

wheels stand upon the span. It is necessary to insert the proviso just made, that at least one wheel shall be upon the span, for otherwise L would vanish and (4) could not change sign. This proviso necessarily excludes from consideration by this method the panel at the right extremity of the span in which S is invariably zero or negative, for when W_1 stands at its right-hand panel point L vanishes. But that is a matter of no consequence, for the greatest shear in the end panel has been otherwise found before.

In Fig. 7, as already shown, the train can only enter upon the span to a distance equal to that between W_1 and W_2 before the maximum value of S will be dominated by W_2. Consequently $A_1 B_1$ represents the extreme position of the span when two wheels and no more have entered as far as possible upon the span. Hence $C B_1$ is the segment dominated by W_1 after leaving out one panel length at the end where, as before mentioned, the shear is either zero or negative. But $C B_1$ is less than a panel length, consequently no panel of this truss is actually dominated by W_1.

Next let $P = W_1 + W_2 = i i_2$, and prolong $k_1 i_2$ to j_2, then by similarity of triangles we have $m j_2 = (W_1 + W_2) \frac{l}{p}$; and as before, when any load not greater than $m j_2$ has entered upon the span, the maximum value of S is dominated by W_2 in all that part of $C B_2$ not already shown to be dominated by W_1. In Fig. 7 this includes all the remaining panels, for they stand at the left of panel points dominated by W_2. This, however, does not prevent the possibility of a shear still greater than this in the last panel at the left, after one or more wheels have left the span, which will be found by the method previously explained in connection with Fig. 2.

None of these constructions require lines to be drawn upon the permanent sheet, as the distances $C B_1$, $C B_2$, etc., can be read off on the cross-section paper when a stretched thread has been put successively in the positions $k_1 j_1$, $k_1 j_2$, etc.

8. The position of a train when it produces a maximum bending moment at any point C of the span $A B$ may also be readily found by help of the weight line alone, as will now be shown.

It is to be noticed that the determination of the maximum bending moment at any point C of the span $A B$ will suffice to determine not only the maximum moment at that point of a simple girder but also the maximum stress in that member of the chord of a single intersection truss which is opposite to any pin point C which carries the live load.

Suppose the live load to consist of one or more locomotives with tenders, followed by an unlimited uniform train, and preceded or not, as the case may be, by a uniform train.

The expression for the value of M, the bending moment at C, may be written thus :

Digitized by Google

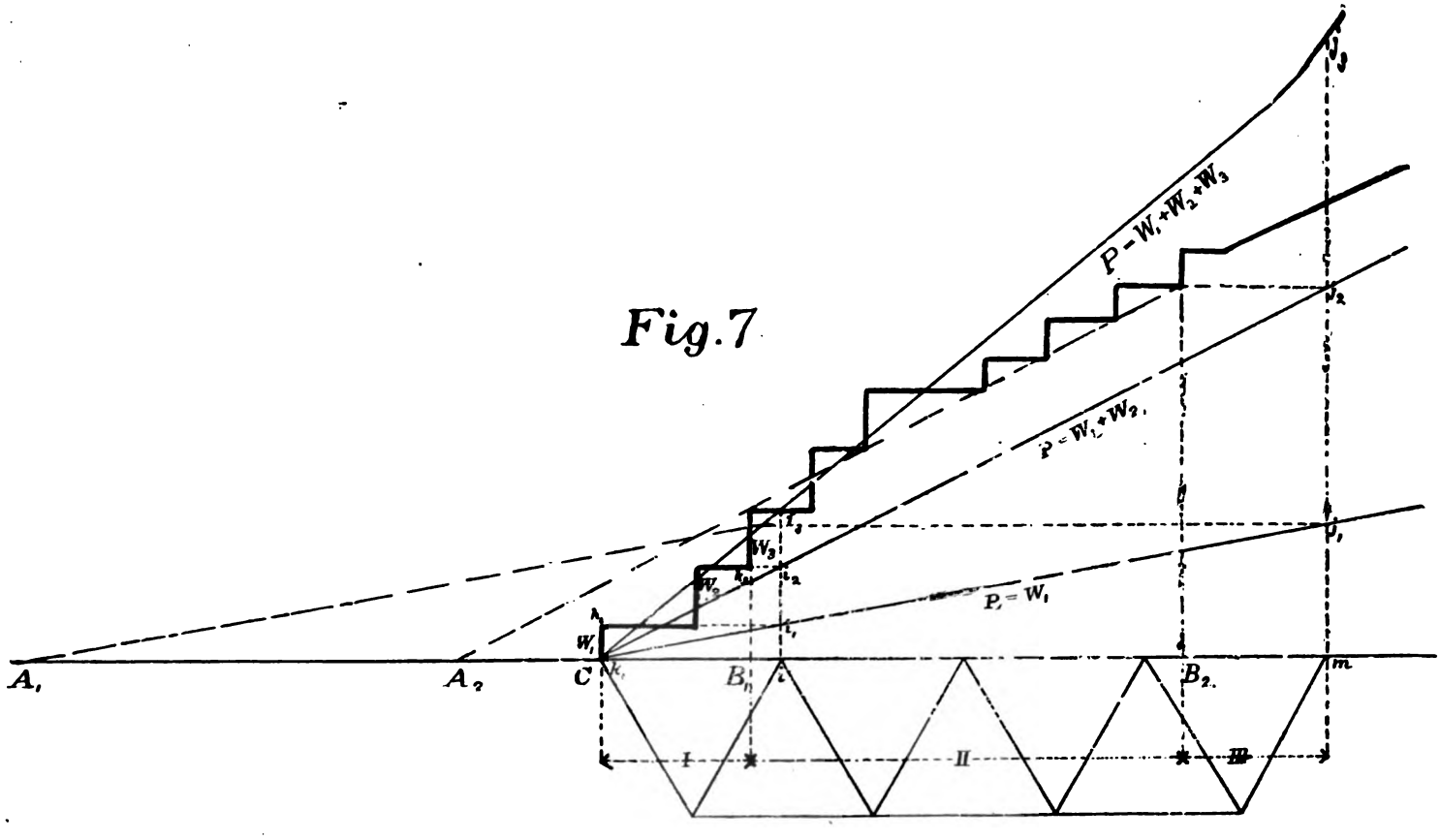


Fig.7

Digitized by Google

$$M = \frac{c_a}{l} \sum_{A}^{B} W b - \sum_{A}^{C} W c,$$

in which equation c_a is the distance from C to A, b is the distance from B to any wheel weight W upon $A\ B$, and c is the distance from C to any weight W upon $A\ C$.

Now let the train move an indefinitely small distance $\delta\ x$ to the left and we have

$$M + \delta\ M = \frac{c_a}{l} \sum_{A}^{B} W (b + \delta\ x) - \sum_{A}^{C} W (c + \delta\ x),$$

from which equation by subtracting the previous expression we obtain a result which may be written in the form

$$\frac{\delta\ M}{\delta\ x} = L \frac{c_a}{l} - C_a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

an equation in which L denotes the total load upon the span $A\ B = l$, and C_a that upon the segment $A\ C = c_a$.

In order that a maximum value of M may occur, the last member of equation (5) must change from positive to negative. Such a change can only occur, during the movement towards the left, by the passage of some wheel weight W_c from the right to the left of the point C, which will increase C_a by an amount W_c.

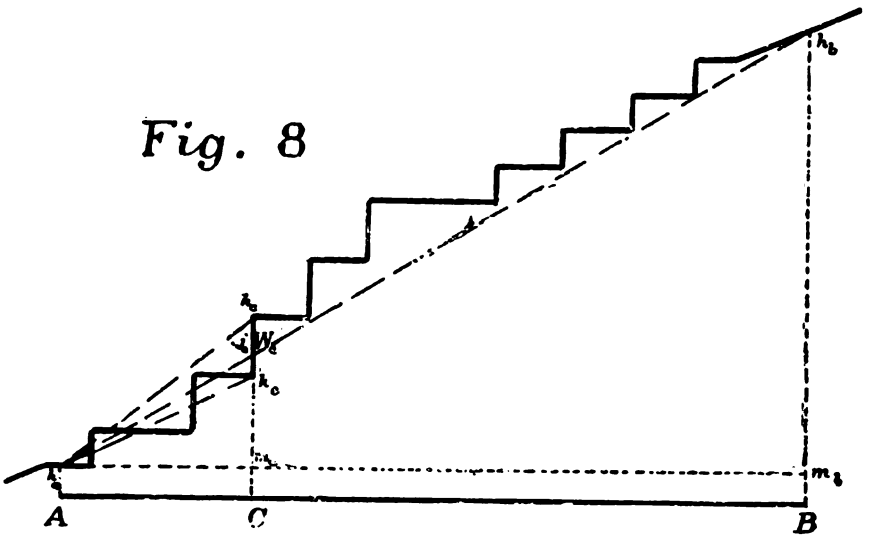
That the change can occur in this manner is sufficiently evident, but that it can occur in this manner alone may be seen by considering what changes of sign the last member of equation (5) undergoes when wheels either enter upon the span or depart from it. When any wheel enters upon the span at B, its effect is to increase L; but an increase of L cannot cause (5) to become negative. Likewise, when any wheel leaves the span at A, the effect of its departure upon (5) is to diminish both L and C_a by the amount of the wheel weight and thereby on the whole to increase (5), for since $c_a < l$ the departure of the wheel has more influence in causing a numerical diminution of C_a than of $L \frac{c_a}{l}$.

For example, in Fig. 8, for the given position of the span $A\ B = l$, we find the total load $L = m_b\ h_b$ and by similar triangles $L \frac{c_a}{l} = m_c\ j_c$ and C_a is either $m_c\ k_c$ or $m_c\ h_c$ according as W_c is taken to stand just at the right or left of C. Hence the last member of (5) changes from $+$ to $-$ as W_c passes C, and W_c when it reaches C causes a maximum value of M at that point.

In order to find in practice which wheel or wheels produce a maximum moment at C, mark the points $A\ C\ B$ upon the edge of a card, and placing C successively under the wheels upon the permanent sheet, stretch a thread from the point h_a to the point h_b, which points are found directly above A and B, provided $A\ C\ B$ is horizontal. If the

Digitized by Google

Fig. 8



thread $h_a\ h_b$ crosses the wheel-weight under consideration, then is the moment at C a maximum. If the thread passes entirely above that part of the weight-line in the vertical at C shift the span $A\ C\ B$ toward the right until it cuts the weight at C; if it passes below, shift $A\ C\ B$ toward the left until the thread cuts the weight at C. The method just described for finding the position of the train which produces a maximum moment, was first proposed and taught, so far as is known to the writer, in 1871, by Professor Mohr, then at Stuttgart, now at Dresden, and the useful device of a thread in place of lines is due to Ward Baldwin, C.E. The method has proved to be so useful and expeditious in practice, that the present writer has been induced to make an endeavor to enlarge the circle of our knowledge as to the possible applications of similar graphical constructions, and this present paper is the outcome of his endeavors.

It may frequently be convenient to find the segment of the span in which the maximum bending moment is dominated by a given wheel, say W_c.

This is easily accomplished by supposing C to remain in the vertical at W_c, while the span $A\ B$ is shifted in one direction until $h_a\ h_b$ passes through k_c or k_c, and then in the other direction until it again passes through k_c or k_c. The part of the span $A\ B$ which passes C in shifting $A\ B$ from one of these extreme positions to the other, is the segment dominated by W_c, and may be denoted for convenience by the same number as the wheel.

Digitized by Google

These segments will overlap each other, *i. e.*, there will often be more than one wheel which causes a maximum moment at a given point.

Which of these maxima is the greater can be discovered by the help of the reaction polygon as a case of the following more general problem.

9. To determine which of two given positions of the train produces the greater bending moment at any given point C of the span $A\,B$.

Let the points $A\ C\ B$ be taken at first in the position $A'\ C'\ B'$, as shown in Fig. 9; then the bending moment at C' may be written with the same notation, as before in the form.

$$M' = \frac{c_a}{l} \sum_{A'}^{B'} W b' - \sum_{A'}^{C'} W c'$$

in which equation c_a is the distance from C to A, or from C' to A'. Now let the train be supposed to move a distance x to the left, then will the points $A\ C\ B$ be shifted a distance x towards the right upon the diagram to the position $A''\ C''\ B''$ such that $A''\ A' = B''\ B' = C''\ C' = x$.

The moment M'' at C'' may be written:

$$M'' = \frac{c_a}{l} \sum_{A''}^{B''} W b'' - \sum_{A''}^{C''} W c''$$

Then do the following equations hold for each wheel:

$$b'' - b' = x,\ c'' - c' = x.$$

Change the limits in M'' by help of the identities

$A''\ B'' = A'\ B' - A\ A'' + B'\ B''$,

$A''\ C'' = A'\ C' - A'\ A'' + C'\ C''$, and subtract M' from it, etc.

$$\therefore\ M'' - M' = \frac{c_a}{l} \left[\sum_{A'}^{B'} W\,(b'+x) - \sum_{A'}^{A''} W\,b'' + \sum_{B'}^{B''} W\,b'' - \sum_{A'}^{B'} W\,b' \right] - \left[\sum_{A'}^{C'} W\,(c'+x) - \sum_{A'}^{A''} W c'' + \sum_{C'}^{C''} W\,c'' - \sum_{A'}^{C'} W\,c' \right]$$

Cancel terms in b' and c', and in the summations between the limits A' and A'' put

$$b'' = l + a'',\ \text{and}\ \ c'' = c_a + a'',$$

in which last two equations a'' denotes the distance of any wheel to the left of A''. Hence, a'' is positive for all wheels between A' and A''.

$$\therefore\ \frac{M'' - M'}{x} = \left[\sum_{A'}^{B'} W - \sum_{A'}^{A''} W\frac{a''}{x} + \sum_{B'}^{B''} W\frac{b''}{x} \right] \frac{c_a}{l} - \left[\sum_{A'}^{C'} W - \sum_{A'}^{A''} W\frac{a''}{x} + \sum_{C'}^{C''} W\frac{c''}{x} \right]$$

Or $$\frac{M'' - M'}{x} = \left(L' - U_{a'} + U_{b'} \right) \frac{c_a}{l} - \left(C_{a'} - U_{a'} + U_{c'} \right) \ldots\ (6)$$

Digitized by Google

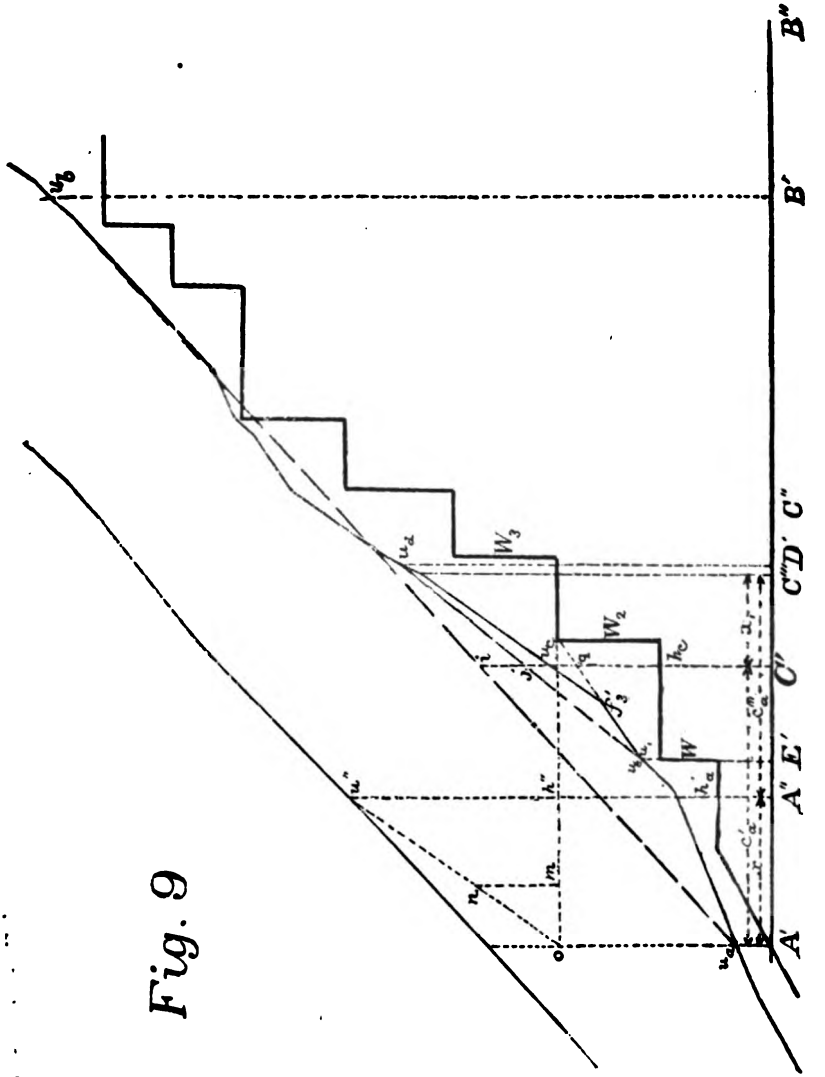


Fig. 9

Digitized by Google

in which equation the quantity

$$U_a' = \sum_{A'}^{A''} W \frac{a''}{x}$$

is the reaction at the extremity A' of the wheel weights upon the span $A'A''$ of length x. Similarly U_b' and U_c' are the reactions at B' and C' respectively of the load in the spans $B'B''$ and $C'C''$, each of length x.

Hence if $u_a\ u_c\ u_b$ be the reaction polygon for the span x, the altitude of u_a above u_b will represent the magnitude of the first parenthesis in equation (6), and by similarity of triangles the altitude of i above u_a will be that part of the second member of (6) preceding the second parenthesis.

It likewise appears that the second parenthesis is the altitude of u_c above u_a. Hence, finally, equation (6) may be written

$$\frac{M'' - M'}{x} = u_c\ i,$$

from which we see that when u_c lies above $u_a\ u_b$ then $M'' < M'$, but when u_c lies below $u_a\ u_b$ then $M'' > M'$. This will enable us to discover which of two maxima values of M is the greater.

It will be further necessary to investigate the position of the train which will produce a maximum stress in the bars of that chord which carries the live load. To this we now proceed.

10. The position of the train for a maximum stress in any chord member $D\ E$ of a single intersection truss $A\ B$, when D and E are pin points of the chord which carries the live load, may be readily found by help of the load line alone, thus:

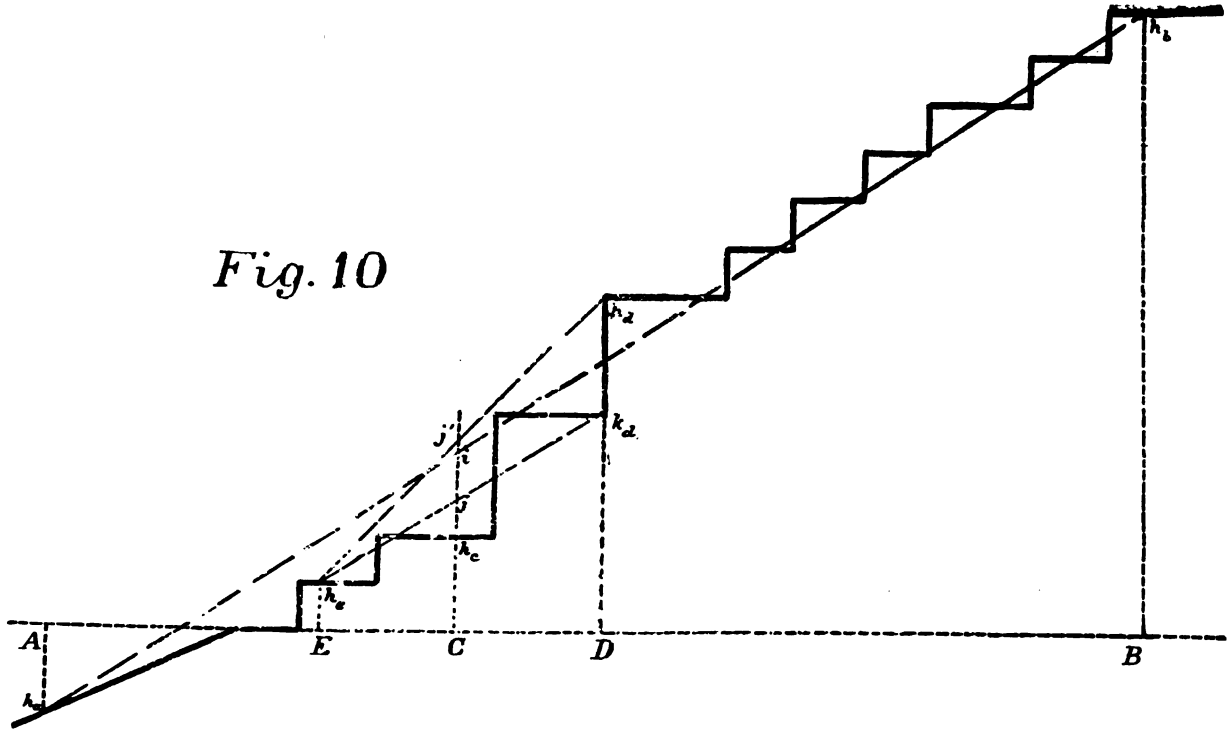
In Fig. 10 let $D\ E$ represent one panel of length p of the single intersection truss $A\ B$, whose span is l, and let the pin point opposite $D\ E$ be in the vertical at C. Denote the distance $C\ A$ by c_a and $C\ E$ by c_e. Also let the distance from B to any wheel in $A\ B$ be b and from D to any wheel in $D\ E$ be d. Then will the bending moment M, which determines the stress in the bar $D\ E$, be the difference between the total bending moment at C, due to the all wheel weights in the span $A\ B$, and the bending moment at C, due to the wheel weights in the panel $D\ E$, for the bending moment due to the latter is borne by the stringers.

$$\therefore \quad M = \frac{c_a}{l} \sum_{A}^{B} Wb - \sum_{A}^{C} Wc - \left[\frac{c_e}{p} \sum_{E}^{D} Wd - \sum_{E}^{C} Wc \right]$$

Let the train move an indefinitely small distance δx toward the left

Digitized by Google

Fig. 10



Digitized by Google

and we obtain in the same manner as before

$$\therefore \quad \frac{\delta M}{\delta x} = \frac{c_a}{l} \sum_{A}^{B} W - \sum_{A}^{C} W - \left[\frac{c_e}{p} \sum_{E}^{D} W - \sum_{E}^{C} W \right] \ldots . \quad (7)$$

The last member of (7) must change from positive to negative in order that M may reach a maximum value.

It is clear from the discussion previously had respecting equation (5) that such change cannot occur by reason of wheels entering upon or departing from the span.

Furthermore, it will be found by considering equation (7) that no finite change occurs when a wheel passes C.

Again, whenever a wheel leaves the panel at E, both of the summations in the parenthesis of (7) will be decreased by that wheel weight, but that will have a greater effect upon the last term of the parenthesis than upon the first term, because $c_e < p$, and hence have an effect upon the whole to diminish the second member of (7) and render it negative. Hence a maximum stress in $D\,E$ may be due to the fact that some wheel leaves the panel at E. Also, whenever a wheel enters upon the panel $D\,E$ at D, that may change the sign of the last member of (7) from positive to negative, for it will have no effect except to increase the first term of the parenthesis. Hence a maximum stress in $D\,E$ may exist when some one of the wheels stands at D, or, as has just been shown, at E.

Now, in Fig. 10 let D lie in the vertical of some wheel Wd, and let h_a, h_e, h_c, h_b be the panels of the weight line in the verticals at A, E, C, B, respectively, and let the vertical at C intersect the slope lines $h_a\ h_b$, $h_e\ h_d$ and $h_e\ k_d$ at i, j' and j respectively, then the first two terms of the last member of (7), as has been before shown, are equal to $k\ i$, and the parenthesis of (7) is equal to $h_c j'$ or to $h_c j$, according as Wd stands upon $D\,E$, or does not stand upon $D\,E$. Hence the last member of (7) changes from + to − when Wd enters the panel at D in case i lies between j' and j. And in general when any wheel stands at D or E such that the point i at the point of intersection of the slope line $h_a\ h_b$ for the span and the vertical at C lies between the two slope lines for the panel, then is the train in a position to produce a maximum stress in $D\,E$.

It will be found quite unnecessary in practice to draw any of the span or panel slope lines upon the permanent sheet.

11. There may evidently be several positions of the train which command a maximum stress in $D\,E$ in the manner just shown. In order to find which one produces the greatest stress in $D\,E$, we shall discuss the more general problem, to discover which of two positions, taken arbitrarily, produces the greater stress in $D\,E$.

Let the quantities M, b, c and d, as used in last discussion, be denoted by M', b', c' and d' in the left of the two positions, and by M'', etc., in the right position, and let x be the distance between the two positions,

Digitized by Google

just as before in Fig. 9. Now, changing limits, performing subtraction and reductions like those employed in obtaining equation (6), we obtain from the first equation of the last article an equation in which appear not only all the terms which are found in equation (6) having reference to the points $A\ C\ B$, but also an equal number of similar terms with opposite signs having reference to the point $E\ C\ D$. The result may be written in the form

$$\frac{M''-M'}{x} = (L'-U_a'+U_b')\frac{c_a}{l} - (C_a'-U_a'+U_c') - \left[(P'-U_e'+U_d')\frac{c_e}{p} - (C_e'-U_e'+U_c')\right] \ldots\ldots\ldots (8)$$

in which equation P' is the total load upon the panel in the position $D'\ E'$, and C_e' is that upon $C''\ E''$, while U_c', U_d', U_e' are the reactions at the left extremities respectively of the spans $C'\ C''$, $D'\ D''$, $E'E''$, each of length x.

The first half of the last member of (8), referring to the points $A'\ C'\ B'$ is expressed, as has been already shown by the vertical distance $u_c\ i$ in Fig. 9; while the last half of it, referring to the points $E''\ C'\ D'$, as expressed by the vertical distance $u_c\ j$, provided u_a, u_e, u_d, u_b is the reaction polygon of the span x. Hence it appears that when j lies above i, $M'' < M'$, but when j lies below i, $M'' > M'$. The application of this method will enable us to discover which of the two positions, each of which produces a maximum stress in $D\ E$, produces the greater maximum.

We here omit, as of comparatively little practical interest, a comparison of the stresses in any two bars of the chord which carries the live load in a single intersection truss under any assumed positions of the load, such, for example, as produce maximum stresses in those bars. The method bears the same relation to that developed from equation (8), as that of (9) does to (6).

12. This part of the discussion will be concluded by the consideration of a somewhat more general problem: to discover whether the bending moment at one point C' of the span in one position $A'\ B'$ is greater or less than the bending moment at any other point C''' of the span, when the span is shifted to any other position $A''B''$.

Let x_1, Fig 9, denote $C'''C'$ the distance along the train between the points at which the moments are taken, and as before let x denote the distance $A''A' = B''B'$ through which the span is shifted; then is $x-x_1 = C''C'''$ the distance along the girder between the points where the moments M''' and M' are taken. Also let $C'\ A' = c_{a'}$ and $C'''A'' = c_{a'''}$. Then, as before, the moment at C' may be written:

$$M' = \frac{c_{a'}}{l}\sum_{A'}^{B'} W b' - \sum_{A'}^{C'} Wc';$$

and that at C''':

Digitized by Google

$$M''' = \frac{c_a'''}{l} \sum_{A''}^{B''} W b'' - \sum_{A''}^{C'''} W c'''.$$

Also $c_a' + x_1 = c_a''' + x,$

or $c_a''' = c_a' - (x - x_1),$

and $b'' - b' = x = a'' - a',\ b'' = a'' + l,$

$c''' = c_a''' + a'' = c' + x_1 - x + a'',$

$c''' = c + x_1 = c'' + x_1 - x,$

$$\therefore M''' - M' = \frac{c_a'}{l}\left[\sum_{A'}^{B'} W b'' - \sum_{A'}^{A''} W b'' + \sum_{B'}^{B''} W b''\right]$$

$$- \frac{x - x_1}{l} \sum_{A''}^{B''} W b'' - \frac{c_a'}{l} \sum_{A'}^{B'} W (b'' - x)$$

$$- \left[\sum_{A'}^{C'} W c''' - \sum_{A'}^{A''} W c''' + \sum_{C'}^{C'''} W c''' - \sum_{A'}^{C'} W (c''' - x_1)\right].$$

Cancel two pairs of terms, substitute values of c''', etc., rearrange, and we have

$$M'' - M' = \frac{c_a'}{l}\left[x \sum_{A'}^{B'} W - \sum_{A'}^{A''} W (l + a'') + \sum_{B'}^{B''} W b''\right]$$

$$- \frac{x - x_1}{l} \sum_{A''}^{B''} W b'' + \sum_{A'}^{A''} W (c_a' + a'' + x_1 - x)$$

$$- \sum_{C'}^{C'''} W (c'' - x + x_1) - (x - x + x_1) \sum_{A'}^{C'} W,$$

from which equation we finally obtain the result:

$$\frac{M'' - M'}{x} = \frac{c_a'}{l}\left[\sum_{A'}^{B'} W - \sum_{A'}^{A''} W \frac{a''}{x} + \sum_{B'}^{B''} W \frac{b''}{x}\right]$$

$$- \left[\sum_{A}^{C'} W - \sum_{A'}^{A''} W \frac{a''}{x} + \sum_{C'}^{C'''} W \frac{c''}{x}\right] + \sum_{C'}^{C'''} W \frac{c''}{x}$$

$$- \frac{x - x_1}{x}\left[\sum_{A''}^{B''} W \frac{b''}{l} - \sum_{A''}^{C'''} W\right] \quad \ldots\ldots\ldots\ldots (9)$$

As previously explained, the first two group terms in the last member of (9) are together expressed by $u_c\ i$ of Fig. 9 in case $u_a\ u_c\ u_b$ is the reaction polygon for the span whose length is x. The last paren-

Digitized by Google

thesis, in equation (9), is the shear at C''' in the span A'' B'', in which the first summation is the reaction at A'' of the total load upon A'' B'', and is expressed by h_a'' u'' in case u'' is upon the reaction polygon for a span of length $l = A''$ B'', while the last summation is the total load resting on A'' $C'' = h_a''$ h'', hence the magnitude of the quantity within the last parenthesis is expressed by h'' u''. Now lay off h'' $o = x$, and h'' $m = x_1$, then by similar triangles it appears that the value of the last group term in (9) is m $n = \frac{x - x_1}{x_1}(h''$ $u'')$. The only term not yet interpreted is the one without the parenthesis. It signifies the reaction at C' of all the wheels resting on the span C'' C'', which are situated between C''' and C', and its value is the segment of ordinate at C', included between u_c and u_1 f_3', that is,

$$\sum_{C''}^{C'''} W\frac{c''}{x} = q \text{ nc}$$

Hence equation (9), of which equation (6) is a particular case, may be interpreted in a manner analogous to (6), thus:

$$\frac{M''' - M'}{x} = q\ i - m\ n$$

When i falls below q then is $q\ i$ negative; and when $x_1 > x$ then will m be at the left of o, n will fall below $m\ n$ and $m\ n$ will be negative.

In the case shown in Fig. 9 it appears that M''' the moment at C''' in the span when situated at A'' B'' is greater than M' at C' in A' B'.

The foregoing demonstration and construction might at first sight appear to be of purely theoretical interest, but it finds important application in determining how many wheels rest on a stringer when it is subjected to the greatest bending moment possible during the passage of the train. For example, if a stringer be of such length as to take only the three of the heaviest wheels at once, it may nevertheless suffer a greater bending moment when only two of them are upon it, by reason of their being nearer the center. Then is x_1 the distance apart of the two wheels under which the maximum moments M''' and M' occur respectively. It will frequently occur that $x_1 = 0$. The point C''' in the span A'' B'' and C' in A' B' where each given number of wheels produces its greatest bending moment can easily be found, as will be seen from the investigation which follows respecting limited sets of wheels, all of which remain upon the span.

In applying the general method of graphically comparing any two bending moments with each other to the case just mentioned (when a wheel stands at the point C'''), it might not at first sight be clear whether this wheel should be included in the summation from A'' to C''' or in that from C'''' to C''. But an examination of the question

Digitized by Google

will show that it makes no difference which summation it is included in, for a variation of x_1 such as so cause C''' to pass a wheel weight and so pass it from A'' C''' to C'' C'' makes no sudden change in the value of (9); for when a wheel is at C''' we have for that wheel $c''=x-x_1$. The value of (9) therefore varies continuously with x or x_1.

13. A few problems will now be discussed which relate to the bending moments in a simple girder carrying a given limited set of wheels at invariable distances from each other, all of which remain upon the girder. These problems will serve especially to display the advantage of using the weight line and certain of its modifications, as applied to the case of a limited set of wheels. The case is necessarily less complicated than that of an unlimited train, treated heretofore, and hence it admits, in some particulars, of a more complete solution than does the more general case.

The first of these problems is to subdivide the span A B into segments I, II, III, etc., in which the maximum bending moment is dominated by the successive wheels W_1, W_2, W_3, respectively, which constitute the set.

Let the wheel weights W_1 to W_4 represented in Fig. 11 all remain upon the span A B; and in any position, assumed at will, draw the slope line h_a h_b. Call the points where it intersects h_1 k_2, h_2 k_3 and h_3 k_4, i_1, i_2 and i_3, respectively.

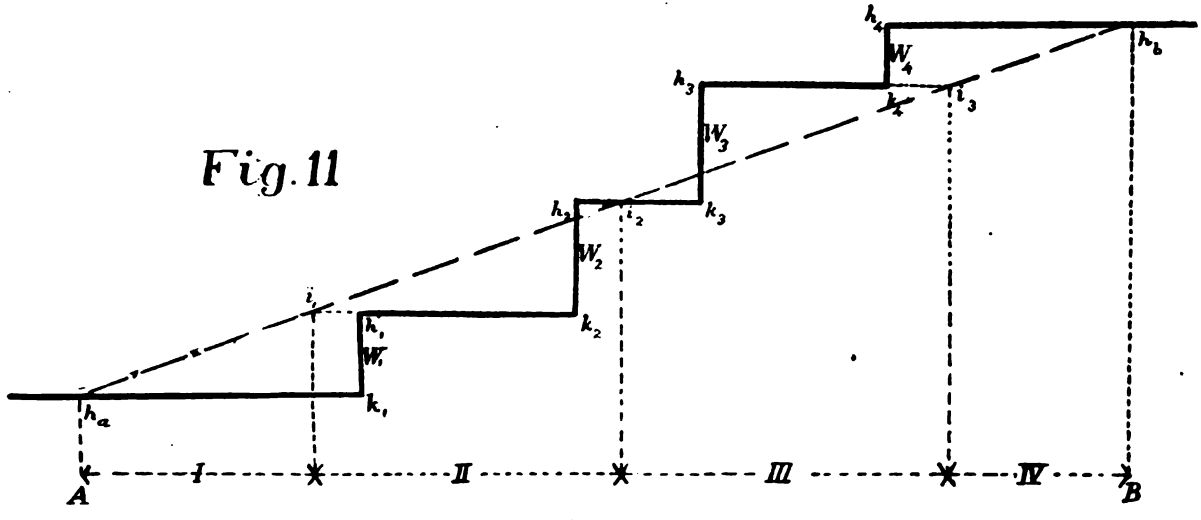
If the set be moved to such a position that W_1 stands at i_1 it is evident that the last member of equation (5) would then vanish in case C were taken at i_1, and in case C were taken at any point of the span at the left of i_1, the last member of (5) would change from positive to negative as W_1 moves from right to left of it. Moreover, if W_2 and C be moved to i_2 the last member of (5) would again vanish, and for any point between i_2 and i_1 the last member of (5) would change from positive to negative as W_2 moves from right to left of it. Hence in segment I, extending from A to the vertical at i_1, the maximum moment is dominated W_1. Similarly segment II extends from the vertical of i_1 to that of i_2, etc.

From similarity of triangles it is evident that the lengths of the segments I, II, III, IV, are proportional to the wheel weights W_1, W_2, W_3, W_4; and in general the lengths of the segments of the span in which the moments are dominated by the successive wheels of the set are proportional to the weights of those wheels, provided only that it is physically possible for each wheel to traverse the entire segment which it dominates without thereby causing any other wheel of the set to move off from the span.

If, for example, segment I is shorter than the distance between W_1 and W_2, then W_2 could not move as far as i_1 before W_1 would leave the span: but W_2 would cause the maximum moment in all that part of segment II which it can traverse while W_1 is upon the span. It would

Digitized by Google

Fig. 11



Digitized by Google

still be an open question as to the remainder of segment II whether the greater moment would be caused at any point of it by W_1 or by W_2 with W_1 off the span, or possibly by some other wheel as it traverses it, but it is a question which could be settled by Art. 9. Similar statements hold respecting any other segment when the entire segment cannot be traversed by its dominating wheel without causing some other wheel to leave the span.

14. To discover the point of the span where each wheel of a given set, all of which remain upon the span, causes a greater bending moment than it does at any other point. The bending moment M at any point C, as seen previously, is expressed by the equation

$$M = \frac{c_a}{l} \sum_{A}^{B} W b - \sum_{A}^{C} W c.$$

Let the point C be taken at that wheel, W_c say, whose wheel position of greatest moment we wish to find; then is c_a the distance from W_c to A. Let the set of wheels move an indefinitely small distance δx toward the left, then

$$M + \delta M = \frac{(c_a - \delta x)}{l} \sum_{A}^{B} W (b + \delta x) - \sum_{A}^{C} W c.$$

Subtract the former of these equations from the latter, and neglect the small term of the second order containing δx^2. The result may be put in the form

$$\frac{\delta M}{\delta x} = \frac{L}{l} \left(c_a - b_g \right) \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

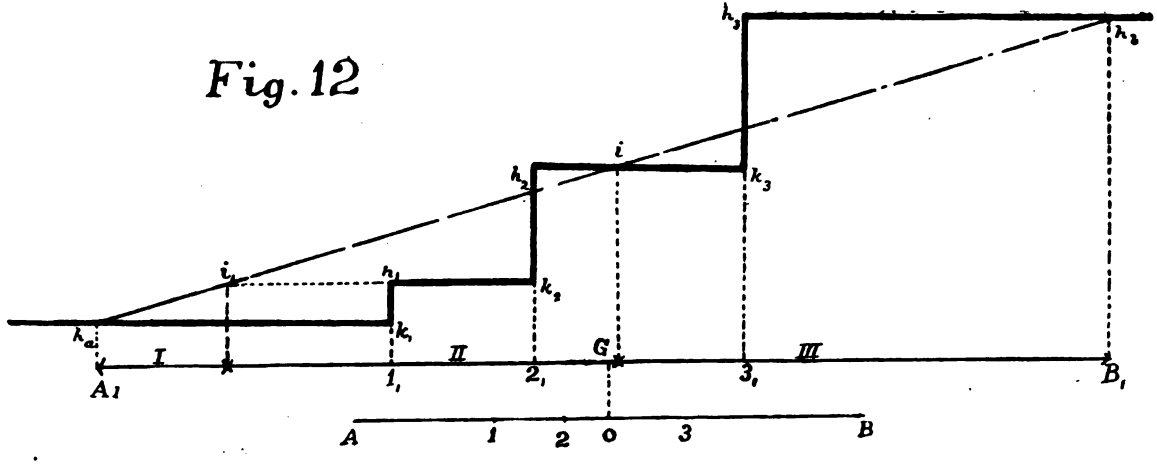
in which equation L denotes the sum total of the wheel weights upon the span, and b_g is the distance of G, their center of gravity, from B. Hence M reaches a maximum when $c_a = b_g$, which shows that the point where any wheel W_c produces its maximum moment is situated as far from A as the center of gravity of the load is from B, or to state it more simply, the wheel W_c and the center of gravity of the load L must be equidistant from the middle of the span in opposite directions.

These two propositions respecting the maximum bending moments due to a limited set of wheels are, it is believed, original with Culmann, who published them in his *Graphische Statik*, First Edition (1866), pages 137 and 138; Second Edition (1875), page 351, but the method of development and presentation was unlike that which has been here employed.

The wheel position of maximum moment for the span $A\,B$ may be constructed very simply by help of the weight line as follows: Let the set of wheels which remain upon the span $A\,B$ be represented by the weight line $k_1 \ldots . h_3$, as shown in Fig. 12, and let G be the position of their center of gravity.

Digitized by Google

Fig. 12



Digitized by Google

It has been shown previously how to find the position of G by using a reaction polygon for any length of span in which these wheels and no others stand. But in case no such reaction polygon has been already constructed, it is to be noticed that the construction for finding the position of the center of gravity of these wheels will be most simple if a span is assumed for that purpose of the same length as the wheel base, so that one extremity is under the first wheel and the other under the last wheel of the set.

Now, having found the position of the vertical at the center of gravity G, it would only be necessary, as has been just shown, to move the train until G is as far from O the middle of the span as W_1 is from O, in order that W_1 should stand in its position of maximum moment. It is evident that W_1 must then stand at 1, a point such that the distance $O1$ is one-half of $G1_1$. Similarly, the point 2 where W_2 produces its maximum moment is situated at a distance $O2$ from O, equal to one-half of $G2_1$, and so on from the other wheels. It will, however, be more simple to lay off $A_1\ G$ and $B_1\ G$ each equal to $A\ B$. Then if $A_1\ B_1$ be taken instead of $A\ B$ to represent the span (on twice the scale of $A\ B$), the point G will be the middle of the span $A_1\ B_1$, and the wheel positions $1_1\ 2_1\ 3_1$ will then be the points where the wheels W_1, W_2, W_3, respectively produce their maximum moments, provided the wheels can reach those points without causing any wheel of the set to leave the span.

15. To discover which wheel of the set produces a greater bending moment in its position of maximum moment than either other wheel produces in its position of maximum moment.

Suppose the span to be subdivided into the segments I, II, III, etc., in which the maximum moment is dominated by the wheels W_1, W_2, W_3, etc., respectively. These segments will be proportional to the weights of the respective wheels, as was shown in Art. 13.

Now, in Fig. 12 it is seen that 1_1 the point where W_1 produces its greatest moment in the span $A_1\ B_1$ lies in segment II, consequently W_2 will cause a greater moment at 1_1 than W_1 does at that point; but W_2 produces a greater moment at 2_1 than it does at 1_1 or any other point of $A_1\ B_1$. From this it is evident that when the point where a wheel produces its maximum moment lies in the segment dominated by that wheel, as 2_1 in segment II, then is the moment at that point greater than that under any wheel whose point of maximum moment lies in the same segment, as W_1 at 1_1 in segment II.

It will be quite unnecessary in practice to actually subdivide the span $A_1\ B_1$ into segments as has been done. The wheel's weight whose points of maximum moment lie within the segments dominated by those wheels are the same wheel weights which are intersected in the weight line by the slope line $h_a\ h_b$. If any one of the wheel weights stands near G, the center of gravity of the set, the slope line will usually intersect that alone, but otherwise it may intersect two successive wheel weights,

Digitized by Google

and we have further to discover in this case which of the wheels produces the greater moment at its position of maximum moment.

In order to discover which of the successive wheels produces the greater moment in the span $A_1 B_1$, in case the weight line is intersected by the slope line for the span at both of these wheel weights in their position of maximum moment, let the equation for the bending moment M' last used be written in the form:

$$M' = \frac{c_a}{l} L b_g - C_a c_g$$

in which equation L denotes the sum total of the wheel weights on $A B$ and C_a those on $A C$, while b_g is the distance from B to the center of gravity of L, and c_g is the distance from C to the center of gravity of C_a, both positive toward the left.

Now introduce the condition for a maximum given by equation (10), viz.: $c_a = b_g$, so that the point C where the moment M' is taken shall be as far from A as the center of gravity of L is from B. The last equation then becomes

$$M' = L \frac{c_a^2}{l} - C_a c_g,$$

in which equation, if c_a be taken as a continuous variable the point C will move in the opposite direction from the train and to an equal amount. Let the train move a distance x toward the left such that no wheel passes C.

Then the moment in the new position may be written:

$$M'' = \frac{L}{l} (c_a + x)^2 - C_a (c_g + 2x)$$

in which equation, since the moving point C has passed no one of the moving wheels, the value of C_a is unchanged.

Hence by subtraction, etc.:

$$M'' - M' = 2 \left(L \frac{c_a}{l} - C_a \right) x + L \frac{x^2}{l}.$$

Now, let the initial position of C corresponding to the moment M' be taken in the vertical at i of Fig. 12, then the last equation reduces to

$$M'' - M' = \frac{x^2}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

for at i we have $$L \frac{c_a}{l} - C_a = 0.$$

Take M'' first at the position of maximum moment on one side of i and then at that on the other side of i.

It is evident that the last member of (11) increases equally for equal + and — values of x; hence $M'' - M'$ and consequently also M'' is greater at that one of the two maximum moment positions which is the more remote from i, provided no wheel leave the span in order to permit this wheel to reach its position of maximum moment.

16. To find where each wheel of a given set, all of which remain on the span, causes its maximum bending moment, in case there is a uni-

Digitized by Google

formly distributed load upon the span in addition to the set of wheel weights. Let w be the weight of the uniformly distributed load per unit of length, and then employing the same notation as before we may write the expression for the bending moment at C in the form:

$$M = L\frac{c_a b_g}{l} - C_a c_g + \tfrac{1}{2} w c_a (l - c_a)$$

Now let C move with the train an indefinitely small distance δx_1 to the left; then

$$M + \delta M = \frac{L}{l}(c_a - \delta x)(b_g + \delta x) - C_a c_g + \tfrac{1}{2} w (c_a - \delta x)(l - c_a + \delta x).$$

After subtracting and neglecting small terms we obtain

$$\frac{\delta M}{\delta x} = \frac{L}{l}(c_a - b_g) + w(c_a - \tfrac{1}{2} l),$$

an equation analogous to (10).

M reaches its maximum value when this expression vanishes. Equate it to zero and the result may be written in this form:

$$\frac{L b_g}{l} = \frac{L + w l}{l} c_a - \frac{w l}{2} \quad \ldots\ldots\ldots\ldots\ldots (12)$$

a result which expresses the relation between c_a and b_g for a maximum moment under a wheel distant c_a from A. We say under a wheel, because it is otherwise known that a maximum can only occur under a wheel. The positions of these points of maximum moment for the successive wheels can be readily discovered as follows:

In Fig. 13 let 1′ 2′ 3′ be the positions of the wheel weights W_1 W_2 W_3 when their center of gravity is at G, the middle of the span $A_1 B_1$, which last is laid off on the same scale as the wheel distances. Let $h_a o = pq = L$ represent the sum total of the wheel weights, and let $om = np = \tfrac{1}{2} wl$ be one-half the total uniform load.

The equation of the slope line $h_a q$ may be written in the form:

$$y' = \frac{L}{l} b_g$$

in case the origin of the co-ordinates b_g and y' is taken at q in the vertical at B, and b_g is reckoned as positive toward the left and y' as positive downwards. Again the equation of the line $m n$ may be written

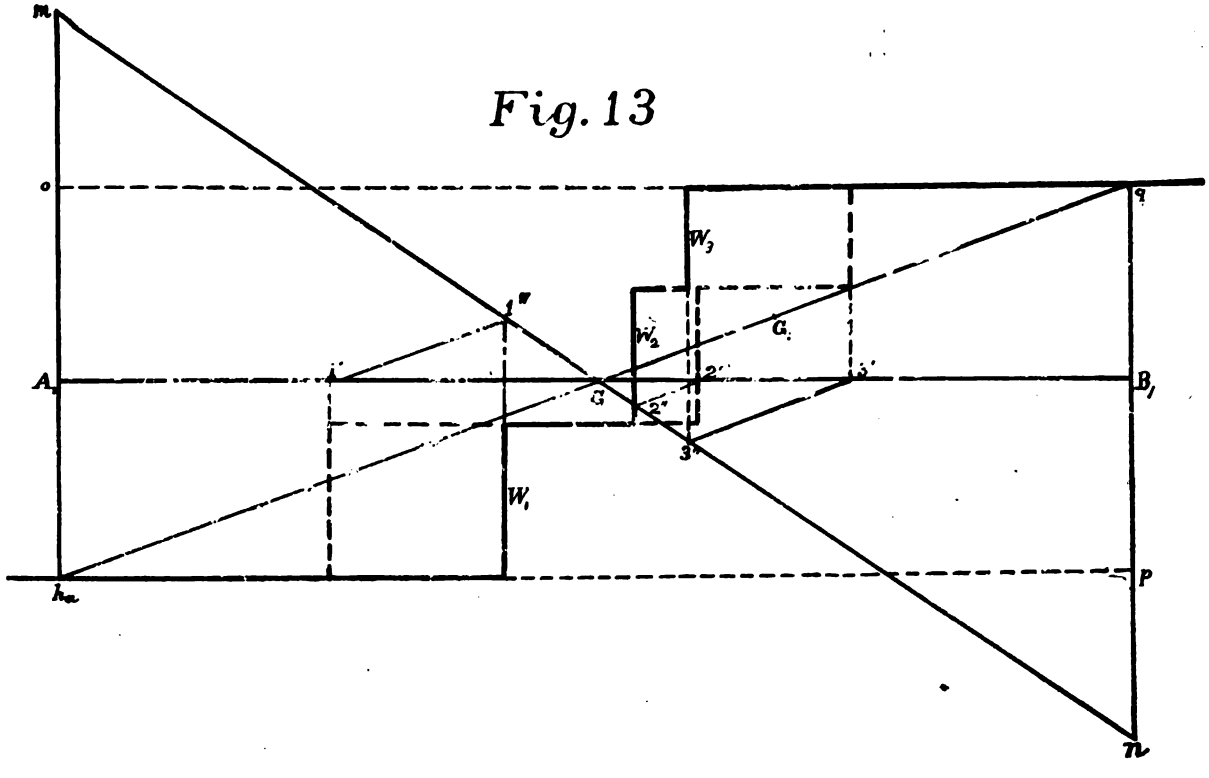
$$y'' = \frac{L + wl}{l} c_a - \frac{wl}{2}$$

in case the origin of the co-ordinates c_a and y'' is taken at o in the vertical at A_1 and c_a is the distance from C to the vertical at A_1 and y'' is positive downwards. In order to have a maximum moment at any point C, we know that C must be taken at a wheel, and furthermore equation (12) shows that c_a and b_g must be so taken that

$$y' = y''.$$

Digitized by Google

Fig. 13



Digitized by Google

To find the point where W_1 fulfills this last condition draw 1′ 1″ parallel to $h_a\,q$, and suppose the set of wheels to have moved to such a position as to have moved W_1 from the vertical at 1′ to that at 1″; then has the center of gravity of the set also moved an equal distance and lies in the vertical at G_1. In case W_1 is at 1′ then is y_1'' the ordinate at 1′ between oq and mn, and y_1' is the ordinate at G_1 between oq and $h_a\,q$. When W_1 reaches 1″ then is y_1'' the ordinate at 1″ between oq and mn, and y_1' is the ordinate at G_1 between oq and $h_a\,q$. Our construction evidently makes $y_1' = y_1''$ in case W_1 is at 1″, and so by equation (12) W_1 produces the greatest moment when it stands at 1″.

Similarly it may be shown that W_2 and W_3 cause their maximum moments in the verticals at 2″ and 3″ respectively, provided 2′ 2″ and 3′ 3″ are parallel to $h_a\,q$. Since the slope line $h_a\,q$ intersects W_2 in its position of maximum moment, it is evident that W_2 in this position satisfies the condition given in equation (5) for a maximum moment at any point of the span, and consequently W_2 at 2″ will produce the greatest moment in the span. But in case the slope line $h_a\,q$ should intersect two successive wheel weights in their positions of maximum moment, it will be further necessary to discover which wheel produces the greater moment of the two.

17. To find which wheel of a given set causes a greater maximum moment than any other, in case there is an additional uniform load.

As just shown, that wheel weight which is intersected by the span slope line in position of maximum moment is the one which produces the greatest maximum moment, provided there is but one such wheel. In case of several such wheels, we might proceed by the general method of Art. 12, but it is much simpler in the usual case where there are only two successive wheel weights intersected by the slope line, to proceed as in Art. 15, which will now be shown to be also correct for this kind of loading.

In the first equation of moments in the last article let the train move a distance x_g to the left, and at the same time let the point C, at which the moment is taken, move to the right a distance x_c, such that while b_g receives the increment x_g, c_a shall receive a corresponding increment, determined by equation (12), thus:

$$\frac{L}{l}(b_g + x_g) = \frac{L+w\,l}{l}(c_a + x_c) - \frac{w\,l}{2}$$

From this subtract (12) and we obtain the relation sought,

$$\frac{L}{l}x_g = \frac{L+w\,l}{l}x_c$$

Now let the train move, as proposed, a distance x_g and we have for the value of M'' the moment at C after it has moved a distance x_c

Digitized by Google

$$M'' = \frac{L}{l}(c_a + x_c)(b_g + x_g) - C_a(c_g + x_c + x_g)$$

$$+ \frac{w\,l}{2}(c_a + x_c)(l - c_a - x_c).$$

Subtract from this the moment M' at C before the movement, then

$$M'' - M' = (L\frac{c_a}{l} - C_a)\,x_g + (L\frac{b_g}{l} - C_a)x_c$$

$$+ \frac{L}{l}x_c\,x_g + \frac{1}{2}w\,l\,(l - 2c_a - x_c)\,x_c,$$

in which equation x_c and x_g have the relation previously obtained, while c_a and b_g have that given in (12). Substitute the value of b_g from (12) and we have

$$M'' - M' = (L\frac{c_a}{l} - C_a)(x_c + x_g) + L\frac{x_c\,x_g}{l} + \frac{1}{2}w\,x_c^2.$$

Take the initial position of C, at which the moment is M', at the point, between the positions of maximum moment of the two successive wheels, at which point

$$L\frac{c_a}{l} - C_a = 0.$$

Then our last equation becomes, after also introducing the value of x_g in terms of x_c :

$$M'' - M' = \frac{L + \frac{1}{2}w\,l}{l}x_c^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

From equation (13) it appears that $M'' - M'$, and, consequently, M'' also, is greater the greater the numerical value of x_c regardless of its sign. Consequently, of two successive wheel weights cut by the slope line while standing in their respective positions of maximum moment, that one produces the greater maximum moment whose position is further removed from i, the point where the slope line cuts the side of the weight line between them.

18. The magnitude of the maximum moments under a set of wheels all of which remain upon the span, can be expressed with great simplicity as the difference between the ordinates of a certain equilibrium polygon and a parabola, which is also an equilibrium polygon.

To show this, suppose the set of wheels $W_1 \ldots W_n \ldots W_r$, whose total number is r, to remain upon the span.

Take the bending moment under any given wheel W_n, and let W_i denote any one of the r wheel weights from W_1 to W_r inclusive.

Then, the expression for the moment may be written

$$M = \frac{c_a}{l}L\,b_g - \sum_{i=1}^{i=n-1} W_i\,c_i$$

Digitized by Google

in which b_g denotes the distance from B to G, the center of gravity of all of the wheel weights L, and the remaining letters have the significations shown in Fig. 14, all distances being taken positive toward the left.

It will be seen that we have,

$$c_i = g_i - g_n, \quad c_n = l - b_n, \quad b_g = b_i - g_i = b_n - g_n$$

By the help of these identities the bending moment may be expressed thus:

$$M = \frac{L}{l}(l - b_n)(b_n - g_n) - \sum_{1}^{n-1} W_i (g_i - g_n)$$

or

$$M = \frac{L}{l} b_n (l - b_n) - \frac{L}{l} g_n (l - b_n) - \sum_{1}^{n-1} W_i (g_i - g_n) \ldots . (14)$$

If we put

$$M_2 = \frac{L}{l} b_n (l - b_n) \ldots . (15)$$

$$M_1 = \frac{L}{l} g_n (l - b_n) + \sum_{1}^{n-1} W_i (g_i - g_n) \ldots . (16)$$

then

$$M = M_2 - M_1 \ldots . (17)$$

It is evident that M_2 is the ordinate of the moment polygon (parabola), due to a single concentrated load L as it moves across the girder, or what is the same thing, it is the moment due to a total uniformly distributed load 2 L. Hence M_2 is easily computed or constructed graphically.

We proceed to show how M_1 may also be constructed graphically, with the greatest ease (though the proof is a trifle tedious), for it will be shown that M_1 is the bending due to the concentrated loads:

$$\frac{L}{l} d_1 \ldots \frac{L}{l} d_n \ldots \frac{L}{l} d_{r-1},$$

acting at fixed points which divide the span into segments having the successive lengths:

$$\frac{l}{L} W_1 \ldots \ldots \frac{l}{L} W_n \ldots \ldots \frac{l}{L} W_r,$$

in which d_1 is the distance from W_1 to W_2, d_n that from W_n to W_{n+1}, etc.

It will be noticed that these concentrated loads are proportional to the successive wheel distances, and their sum is

$$\frac{L}{l} \sum_{1}^{r-1} d = L \frac{l'}{l}$$

Digitized by Google

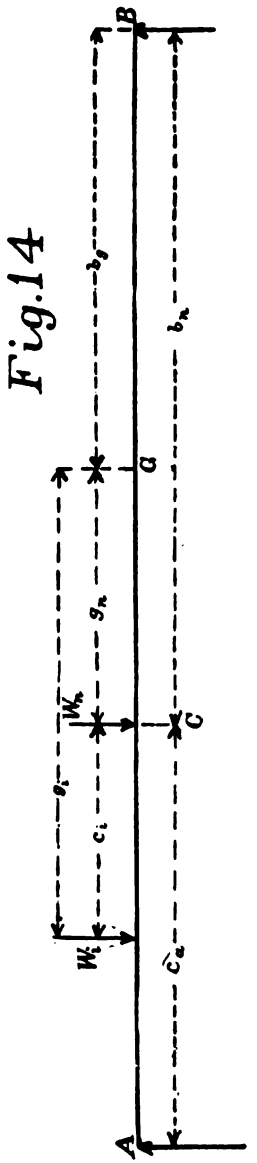


Fig. 14

Digitized by Google

in which l' is the total wheel base, and, further, that the segments of the span l are proportional to the successive wheel weights. Now, taking the moments about B of these proposed weights at their proposed positions, and calling V_1 the reaction at A, we have

$$V_1\, l = d_1 \sum_{2}^{r} W_i + \ldots + d_{n-1} \sum_{n}^{r} W_i + \ldots + d_{r-1}\, W.$$

This quantity can be expressed more simply; for, take the moments of all the actual wheel weights W_2, etc., about W_1, and we have the identity

$$L g_1 = W_2\, d_1 + W_3\,(d_2 + d_1) + \ldots + W_n \sum_{1}^{n-1} d_i + \ldots + W_r \sum_{1}^{r-1} d_i.$$

By rearranging the terms of this equation we find it the same as the moment about B, previously obtained,

$$\therefore V_1\, l = L g_1.$$

Let M_n denote the bending moment due to the proposed concentrated weights at a point of the span distant b_n from B between the weight $\frac{L}{l}\, d_{n-1}$ and $\frac{L}{l}\, d_n$. We then find

$$M_n = V_1\,(l - b_n) - \frac{L}{l}\left[d_1\left(l - \frac{l}{L}\, W_1 - b_n \right) + d_2\left(l - \frac{l}{L} \sum_{1}^{2} W_i - b_n \right) + \ldots + d_{n-1}\left(l - \frac{l}{L} \sum_{1}^{n-1} W_i - b_n \right) \right].$$

Substituting in this equation the value just found for $V_1 = \frac{L}{l}\, g_1$ and rearranging the terms

$$M_n = \frac{L}{l}\left[g_1 - (d_1 + \ldots + d_{n-1}) \right] (l - b_n) + d_1 W_1 + d_2 \sum_{1}^{2} W_i + \ldots + d_{n-1} \sum_{1}^{n-1} W_i.$$

Now $d_1\, W_1 + d_2 \sum_{1}^{2} W_i + \ldots d_{n-1} \sum_{1}^{n-1} W_i$

$$= W_1 \sum_{1}^{n-1} d_i + W_2 \sum_{2}^{n-1} d_i + \ldots + W_{n-1} d_{n-1},$$

$$\therefore M_n = \frac{L}{l}\left(g_1 - \sum_{1}^{n-1} d_i \right)(l - b_n) + W_1 \sum_{1}^{n-1} d_i + W_1 \sum_{2}^{n-1} d_i + \ldots + W_{n-1}\, d_{n-1}$$

Digitized by Google

But
$$d_{n-1} = g_{n-1} - g_n,$$
$$\sum_{2}^{n-1} d_i = g_2 - g_n$$
$$\sum_{1}^{n-1} d_i = g_1 - g_n.$$

By help of these equations we have—

$$M_n = \frac{L}{l} g_n (l - b_n) + W_1 (g_1 - g_n) + W_2 (g_2 - g_n) + \ldots + W_{n-1} (g_{n-1} - g_n)$$

or,

$$M_n = \frac{L}{l} g_n (l - b_n) + \sum_{1}^{n-1} W_i (g_i - g_n)$$

from which it appears that $M_n = M_1$, *i. e.*, the bending moment M_1 may be constructed as that arising from the proposed concentrated weights applied at the points proposed.

This proposition respecting M_1 was first demonstrated by the author about 1885, and was communicated by him to Professor Maurice Levy, who published it with due credit, but demonstrated differently, in his great work *La Statique Graphique, IVe partie*, p. 334.

19. The foregoing development is important, as giving in a single graphical construction the maximum moments under a set of wheels throughout the entire span, but is still more important, perhaps, as leading to a new formula for an approximate value of an equivalent uniform load, which formula we shall now deduce.

It has just been shown in equation (17), that the part M_2 of the total bending moment is the same as that which would be caused by a uniformly distributed load of total amount $2L$, *i. e.*, the equivalent charge per unit of the span is

$$w'' = \frac{2L}{l}$$

The other part M_1 of the bending moment may be regarded as due to concentrated loads $\frac{L}{l} d_1$, etc. In case there are three or more of these at distances not excessively unequal, the moment polygon for M_1 differs but little from a parabola. It may in all ordinary cases be taken to coincide very approximately with the equilibrium polygon (parabola) of the sum of these proposed concentrated weights taken as uniformly distributed. Their sum is

$$\frac{L}{l} \sum_{1}^{n-1} d_i = \frac{L}{l} l'$$

in which $\sum_{1}^{n-1} d_1 = l'$ is the length of the load or wheel base. The charge

Digitized by Google

per unit of length of this load when uniformly distributed is

$$w' = \frac{L}{l^2} l'$$

Hence the total equivalent uniform load is approximately

$$w = w'' - w' = \frac{2L}{l}\left(1 - \frac{l'}{2l}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

It is to be noticed that the last term in (18) takes account of the degree of concentration of the set of wheel weights, and it varies from the value zero, in case the set consists of a single wheel (*i. e.*, $l' = 0$) to the value $\frac{1}{2}$ in case the wheel base covers the entire span, with intermediate values when l' is less than l. It would seem most proper to use (18) in the following manner, in order to apply it to one or more locomotives in the middle of an unlimited uniform train. Let E be the *engine excess*, *i. e.*, the amount by which the locomotives all together exceed the weight of an equal length of the uniform train. We can then consider the span to be loaded throughout with the uniform unlimited train causing a charge of w_t per unit of length, and to that must be added the charge due to a total engine excess, E occupying a length l' supposed to be less than l.

The bending moments in the span will then be due to a charge per foot of

$$w = w_t + \frac{E}{l}\left(1 - \frac{l'}{2l}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Objection may be made to taking the uniform train both in front of the engines and behind the locomotives. But in computing bending moments it is not objectionable so to take the loading, for it is on the side of safety and might sometime actually occur. The length l' should be taken as the total distance between the parts of the uniform train before and behind the locomotives.

20. We shall conclude the discussion respecting the stress due to a limited set of wheels, all of which remain on the span, by finding which wheel produces the maximum shear at any point of a simple girder and then deriving a convenient expression for the weight of a single load which, in conjunction with a uniform distribution, will cause a safe approximation to the maximum shear due to the actual concentrations.

In order to compare together the shearing stresses just at the left of the several wheel-weights, consider a set of r wheels $W_1 \ldots W_n \ldots W_r$, of which W_n is any one, all of which remain on the span. Let $L = \sum_1^r W$ denote the total load on the span AB. Then the shear just at the left of any wheel W_n may be written in the form

$$S = \frac{L}{l} b_g - \sum_1^{n-1} W$$

Digitized by Google

in which equation the summation includes all the wheels at the left of W_n. The notation will be understood from Fig. 14, where we have

$$b_g = b_n - g_n$$

Substituting this value of b_g in the previous equation, we may write the shear thus:

$$S = \frac{L}{l} b_n - \left(\frac{L}{l} g_n + \sum_{1}^{n-1} W \right) \ldots\ldots\ldots\ldots (20)$$

In this equation the shear has been separated into two parts, which can conveniently be separately considered.

Write equation (20),

$$S = S_2 - S_1,$$

then the part

$$S_2 = \frac{L}{l} b_n$$

depends solely upon the abscissa b_n of the point in the span where we take the shear, and it increases uniformly from zero at B to L at A.

In Fig. 15 let $A\,B = l$ and $B\,B_o = L$, then $A\,B_o$ has the slope $L \div l$. Through the center of gravity G draw $A_o\,B$, at the same slope, then is $A_o\,A' = L$, and the vertical ordinate $m_1\,n_1$ between these lines at any point has the value S_2 for that point, as shown in Fig. 15 at W_1. S_2 would have the same value at this point of the span whatever wheel may stand at the point. The other part of S is

$$S_1 = \frac{L}{l} g_n + \sum_{1}^{n-1} W,$$

and depends only on the number and weight of the wheels at the left of W_n, and not at all upon the position of the set upon the span.

The first term of S_1 is the ordinate between $A_o\,G$ and $A\,G$, and the second term is the ordinate at any wheel between $A\,G$ and the lower extremity of the wheel weight, W_n. Now G moves with the set, hence g_n is constant. There is, therefore, a constant value, S_1, for each wheel of the set, regardless of its position upon the span.

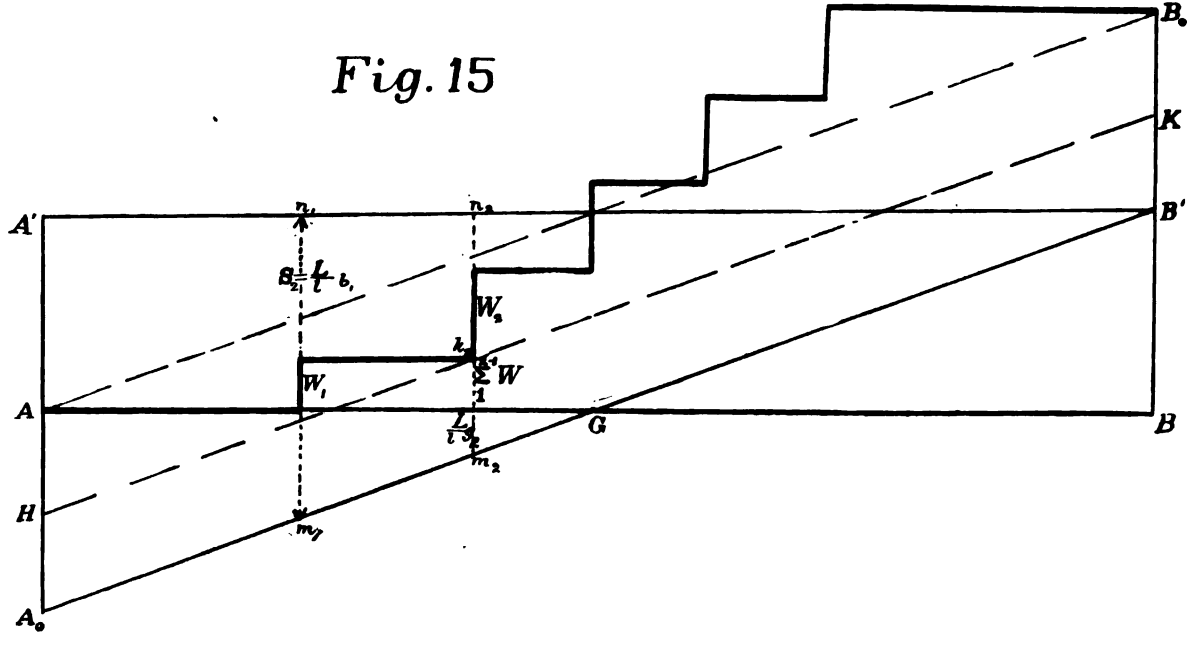
In order that S may have its greatest value at a given point of the span S_1 must have its smallest value. To find the least value of S_1, draw $H\,K$ at the slope $L \div l$ in the lowest position it can be put, and touch the weight line at any point, as at k_2. The vertical ordinate $m_2\,k_2$ between $A_o\,B'$ and $H\,K$ is the smallest possible value of S_1, and the wheel weight which it touches is the one which causes a greater shear than any other so long as all the wheels remain on the span.

$$\therefore S_{max} = m_2\,n_2 - m_2\,k_2 = k_2\,n_2.$$

It has been shown, therefore, that the maximum value of the shear at any point of the span $A'\,B'$ is simply the vertical ordinate at that point between $H\,K$ and $A'\,B'$.

Digitized by Google

Fig. 15



Digitized by Google

21. It is evident that the method just developed, of finding the greatest shear due to a limited set of wheels on the span, may be applied to one or more locomotives followed by a uniform train, in either of two ways. Either add the greatest shear due to the locomotives to that due to the train, or add that due to the locomotive excess to that due to a uniform train whose head is taken to be at W_1.

It will be unnecessary here to discuss that part of the total shear which is due to a train extending without limit toward the right, further than to remark that its amount at any point to the left of the head of the uniform train is

$$S_w = \frac{w\,b^2}{2\,l}$$

in which equation b is the distance from B to the head of the uniform train of which w is the weight per unit of length. S_w added to the shear obtained in equation (20) gives as the expression for the total shear just at the left of W_n, and due both to the locomotives and train:

$$S = \frac{L}{l}\,b_n - \frac{L}{l}\,g_n - \sum_1^{n-1} W + \frac{wb^2}{2\,l} \quad \ldots\ldots\ldots\ldots\ldots \quad (21)$$

in which equation it is understood that all the locomotive wheels are upon the span, and L denotes the weight of the locomotives alone, and b is the length of that part of the uniform train between B and the locomotives.

Again: let us take a different interpretation of (21). Let E denote the engine excess, and let it be superposed upon a uniform train extending from B to W_1 at the head of the locomotives. We may, then, for long spans write as an approximate value of the shear at the left of W_1:

$$S = \frac{b_1}{l}\left(E + \frac{1}{2}\,wb_1\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (22)$$

which will give a shear somewhat in excess of the actual maximum shear for any span long enough to take in all the locomotive wheels. Equation (22) takes the whole engine excess to be situated at W_1 and disregards the second term of (21). The term thus neglected is more than enough to make up for the error committed in taking the shear at the left of W_1 throughout the entire span, instead of taking it just at the left of W_2 or W_3 in the segments dominated by those wheels.

In spans shorter than the wheel base of the locomotives it is evident that the total engine excess is in general less than in the case just considered, though the reverse might occur with a light tender and heavy uniform load, when the tender was not upon the span.

Nevertheless, equation (22) can be used as a useful and safe approximation for short spans as well as long ones.

22. It has been shown in course of the discussion of the properties of the reaction polygon in connection with Fig. 2, that the numerical

Digitized by Google

value of the shearing stress in a girder, or the vertical component of the stress acting in the web members of a single intersection truss, may be found at any point of a given span by simply reading to scale the vertical ordinate at the point between the line of zero shears for the span and the weight line in case of a girder, or between the zero line for the span and the stepped line of zero shears for the panels in case of a truss.

It is, moreover, well known that the numerical value of the bending moment at any point of a girder may be obtained by simply reading to scale the vertical ordinate at the point between the closing line for the span and the equilibrium polygon due to the weights. Such an equilibrium or moment polygon is drawn upon the large diagram, Fig. 3.

The discussion of the moment polygon is a matter somewhat aside from the main purpose of this paper, yet its employment so greatly facilitates practical work that we here give it a brief consideration.

It may seem to the practical bridge designer that the measurement of moments upon such a diagram cannot be made with sufficient accuracy for bridge calculation, but, in fact, perfectly satisfactory readings are now made in the office of the Chief Engineer of the C. N. & T. P. Railway, where this method of reading diagram to scale has displaced that of using tables of moments, which had been employed for years. Of the great convenience and rapidity of the graphical method there can be no question.

A method of constructing the moment polygon and moment lines for each separate wheel weight on profile paper to the scale on which the paper is itself ruled, in such a manner as to avoid all inaccuracies due to drawing parallels, as well as all uncertainties as to scale, and so make the diagram equivalent to a table of moments, has been published by Ward Baldwin, C.E.* This superior method is shown in Fig. 3.

23. To construct and use the moment polygon 1, 2, 3, etc., in Fig. 3, proceed as follows: Take the horizontal line 0 0, which is the zero line from which loads are read, to be also the zero line for moments. At one hundred feet (100′) to the right of wheel 1, lay off from 0 0 a vertical ordinate $= W_1$ pounds. Through its upper extremity and the foot of W_1 draw the moment line 11.

If we take the ordinate at 100′ to represent 100 W_1 foot-pounds, then the scale of moments in foot-pounds is one one-hundredth the scale of weights in pounds, and the ordinate between 0 0 and 11, at any point, is evidently the moment of W_1 about that point in foot-pounds.

Again, at 100′ from W_2 lay off from 11 a vertical ordinate $= W_2$ pounds. Through its upper extremity and the point where 11 intersects the vertical at W_2, draw the moment line 22. Then is the ordinate between 11 and 22, at any point, the moment of W_2 about that point; and the ordinate between 0 0 and 22 at any point is the moment of W_1 and

* *Engineering News*, September 28th, 1889.

Digitized by Google

W_2 about that point. Draw in a similar manner the moment lines 3 3, 4 4, etc.

The uniform train is taken as concentrated in sections of 10′ each at verticals 19, 20, etc., as also at 0, — 1, — 2, etc. These last concentrations give the moment lines — 1 — 1, — 2 — 2, etc. It will be easy to test the accuracy of the construction by computing the moments of the successive wheels about certain fixed points, say at 20′ or 50′ from W_1, and see whether the successive moment lines cross the ordinate at the fixed point at their proper heights. There need be no difficulty in making these constructions to such scale and of such proved accuracy at every point as to merit the full confidence of the practical designer.

These moment lines and the moment polygon formed by them can be employed in several ways, among which we may notice:

First.—The bending moment at any point of a girder is the vertical reading between the polygon and the closing line or thread stretched between those points of the polygon which are in the verticals at the extremities of the span. A scale consisting of a piece cut from the same ruled sheet will be found convenient.

Second.—The bending moment at any pin of the unloaded chord of a single intersection truss is the vertical reading, at that pin, between the closing line for the span and the closing line for the panel.

Third.—The moment of any number of wheels to the left of any point about that point is the reading on the vertical at the point between the moment lines for those wheels. In this manner the value of $\sum_{A}^{B} Wb$ may be read and its value used in computing the shear at any point or in finding center of gravity.

Fourth.—It will be possible to find by actual trial in many cases which one of the wheels produces the greatest bending moment at a given point of the span, especially in case of short spans, by finding at which wheel the ordinate is greatest between the closing line and the moment polygon. In such trials upon the moment polygon a hollow right-angled triangle cut from the same ruled sheet as the diagram will be found to be a great convenience, as also for use in similar trials upon the reaction polygons; for, taking a point upon its horizontal edge as A such that B falls upon the vertical edge, we readily locate the extremities of the span so as to fulfill one condition or another.

24. It is also possible by help of the moment polygon to find in a simple manner those segments of a girder resting on end supports in which the maximum shear is dominated by the successive wheels near the head of the train. This method of solution is due to Ward Baldwin, C. E.,* and is as simple, graphically, as that already given by help of reaction polygons. It is as follows:

* *Engineering News*, October 12th, 1889.

Digitized by Google

Take the expressions already given in Art. 6 for S' and S'' the shears just at the left of C' and C'' respectively, in which the last terms are the total loads to the left of these points. Let C' and C'' be situated at the wheels at which the shears are to be compared. Then the final summations in the expressions for S' and S'' will not include the wheel weights at C' and C'' respectively.

In order to find the point where two wheels produce the same shear, let $S' = S''$, and equating their values as given in Art. 6, transposing, canceling, etc., we have

$$\sum_{A''}^{B''} W b'' - \sum_{A'}^{B'} W b' = l \sum_{C'}^{C''} W \ldots\ldots\ldots\ldots \quad (23)$$

in which $b'' - b'$ for any wheel is the distance which it is necessary to move the train to bring one of the wheels considered to the position previously occupied by the other.

The first term in equation (23) is the height of the moment polygon above 0 0, the zero line of moments at the vertical through B''. The second term is the height of the moment polygon above 0 0 at the vertical through B'. The distance between the verticals at B'' and B' is $b'' - b' = d$, the distance apart of the wheels under which the shears are taken as equal. Hence the first member of equation (23) is the difference in height of two points on the moment polygon whose horizontal distance is d.

The second member shows how much this vertical height must be, for it is the product of the weight of each wheel between C'' and C' multiplied by l, and the product for each wheel can be read off upon the diagram at a distance l to the right of each of those wheels.

Hence to find the point in the span where W_1 and W_2 produce the same shear, read off $l\ W_1$ on the diagram and lay it off downward from the upper left corner O of a rectangular card or separate slip of the ruled sheet, call it Om. Lay off horizontally from the same corner $On = d$, then are Om and On the vertical and horizontal distances between the points of the moment curve at B' and B''. Place the card so that m falls upon the moment polygon, and slide m along the polygon, keeping the card horizontal until n also is upon the moment polygon.

Then m is in the vertical at B' and n in that at B'', such that the shear under W_1 is the same when the right end of the span is at B'. as it is under W_2 when the right end of the span is at B''. The segment dominated by W_1 is, therefore, the distance from W_1 to B', or from W_2 to B'', in the case when this condition is fulfilled. The point where W_2 W_3 produce the same shear may be found in similar manner.

This demonstation supposes that no wheels pass off the span, for had some wheels left the span then some other line than 0 0 would be the line of zero moment.

Digitized by Google

PART II.

MAXIMUM STRESSES UNDER CONCENTRATED LOADS IN CANTILEVERS WITH TWO SUPPORTS; AND IN DOUBLE INTERSECTION TRUSSES WITH END SUPPORTS.

1. The cantilever $E\,D$, Fig. 1, is supposed to be continuous between its extremities E and D and to rest upon supports at A and B, while the spans $F\,E$ and $D\,G$ are supported at their extremities F and G respectively by piers, but at their other extremities, D and E, they are supported by the cantilever itself.

It will be convenient in this discussion of the cantilever to employ a systematic notation like that previously employed, viz.:

Let a denote the distance of any wheel from A, b its distance from B, c from C, etc., etc., and let the distance a be taken as positive when the wheel is at the left of A, but negative when it is situated to the right of A. Similarly, let the distances of a wheel from any other points, as B, C, etc., be taken to be positive toward the left. Also let l_a denote the span $E\,A$ at the left of A, and L_a the total load on the span l_a, and similarly for the other spans at the left of the points B, D and E, with their loads as shown in Fig. 1. Then shall we have the identities:

$$b - a = l_b, \quad d - b = l_d, \text{ etc., etc.}$$
$$d - a = l_b + l_d, \quad e - d = l_a + l_b + l_d, \text{ etc., etc.}$$
$$c - a = c_a, \quad c - e = l_a + c_a, \text{ etc., etc.}$$

2. The position of the train when it produces the numerically greatest shear at any point C of the end span $B\,D$ of the cantilever $E\,D$, in Fig. 1, may be investigated in a manner similar to that previously employed with the simple girder, thus:

It is well known, and need not be here demonstrated, that the greatest shear at any point C in $B\,D$ will be under one of the wheels near the head of the train when the train extends from G to near C, the point considered. In particular the case need not be investigated of a train extending from any point in the opposite direction as F or A and extending up to C; or what is the same thing, it is unnecessary to investigate the shear at points of $E\,A$ when the train extends from A or G or from any point between A and G up to or near the point of $E\,A$ under consideration; for the shear in $B\,D$ is greatest when the train approaches from G, and greatest in $E\,A$ when it approaches from F.

Let V_d be that part of L_g which is supported by the extremity D of the cantilever, then the shear S just at the left of any point C of $B\,D$ is

Digitized by Google

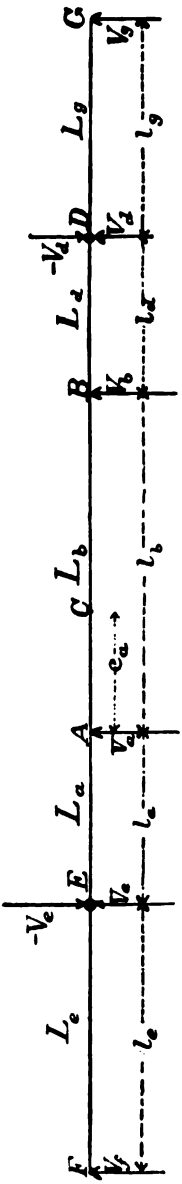


Fig. 1

Digitized by Google

$$S = V_d + \sum_{C}^{D} W. \quad \text{But} \quad V_d\, l_g = \sum_{D}^{G} Wg,$$

$$\text{hence,} \quad S = \sum_{D}^{G} W \frac{g}{l_g} + \sum_{C}^{D} W,$$

in which equation the sign of S is taken as positive when produced by a force acting upward upon the cantilever at the left of C or downward at the right of C, and *vice versa*.

In order to compare the shear under one wheel standing at C with that under some other wheel at a distance x to the right of it when it also arrives at C, let S' denote the shear under the first wheel when at C, and S'' that under the other wheel when at C. Let the train be stationary, and suppose the cantilever to be shifted toward the right a distance x under the train from its initial position, in which C is at C', to the final position in which C is at C''. Denote the corresponding positions of D by D' and D'', and so on for the points A, B, etc.

We then obtain by subtraction :

$$S'' - S' = \left[\sum_{D'}^{G'} - \sum_{D'}^{D''} + \sum_{G'}^{G''}\right] W \frac{g''}{l_g} - \sum_{D'}^{G'} W \frac{(g'' - x)}{l_g}$$

$$+ \left[\sum_{C'}^{D'} - \sum_{C'}^{C''} + \sum_{D'}^{D''}\right] W - \sum_{C'}^{D'} W,$$

in which equation the summations in the first square bracket are together equivalent to a single summation from D'' to G'', and those in the second to one from C'' to D'' by reason of the identities :

$$D''\ G'' = D'\ G' - D'\ D'' + G'\ G''$$
$$C''\ D'' = C'\ D' - C'\ C'' + D'\ D''$$

Also $g' = g'' - x$.

Cancel and make use of the equality $g'' - d'' = l_g$, and we have :

$$\frac{S'' - S'}{x} = \frac{1}{l_g}\left[\sum_{D'}^{G'} W + \sum_{G'}^{G''} W \frac{g''}{x} - \sum_{D'}^{D''} W \frac{d''}{x}\right] - \frac{1}{x} \sum_{C'}^{C''} W \ldots . \quad (1)$$

an equation which admits of a simple geometrical interpretation as shown in Fig. 2.

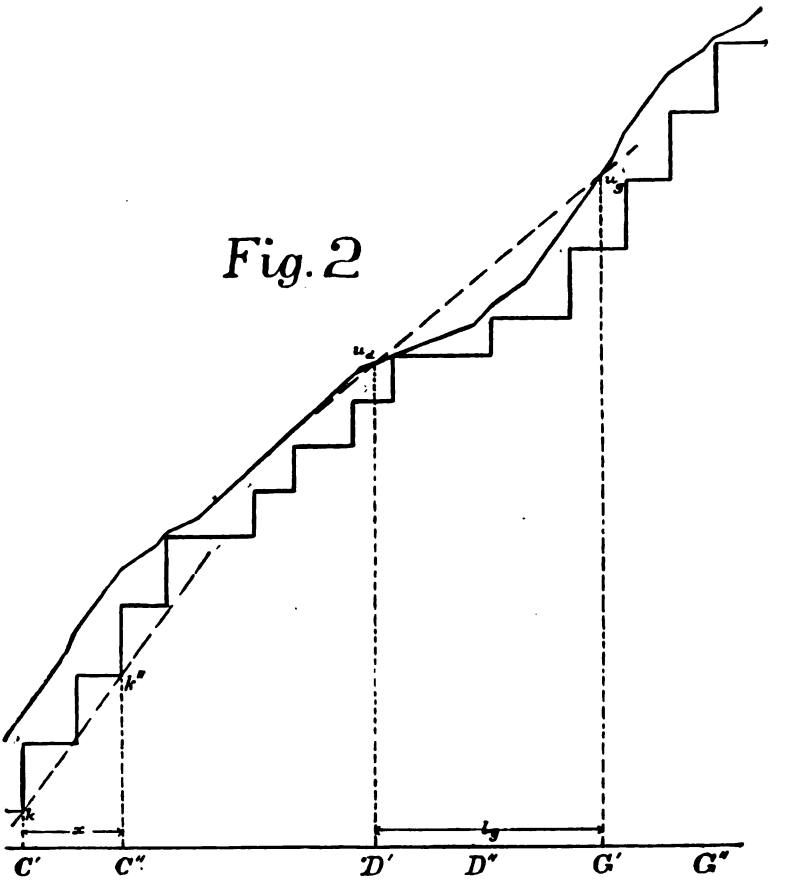
Let u_d and u_g lie on the reaction polygon for the span whose length is x ; then the first group term in the last member of equation (1) is the tangent of the inclination of the slope line $u_d\ u_g$ to the horizontal, and the last term of equation (1) is the tangent of the inclination of the slope line $k'\ k''$ for the load in the span $C'\ C''$.

When the slope of $u_d\ u_g$ is greater than that of $k'\ k''$, then is $S'' > S'$, and *vice versa*.

It is to be noticed that the slope of $u_d\ u_g$ will not greatly vary as an ordinary train moves across the span, especially when l_g is large, and

Digitized by Google

Fig. 2



consequently one and the same wheel near the head of the train will usually dominate the shear in the whole of $B\,D$.

3. To discover what position of the train produces the greatest shear at any point C of the middle span $A\,B$ of the cantilever $E\,D$, Fig. 1, the procedure is similar to that just detailed: The shear S at C is given by the equation

$$S = V_a - V_e - \sum_{E}^{C} W$$

By taking moments about B we find in notation already explained

$$V_a\, l_b = \sum_{E}^{D} W\, b - V_a\, l_d + V_e\,(l_a + l_b)$$

Digitized by Google

Now,

$$V_d\, l_g = \sum_{D}^{G} W g,$$

and

$$V_e\, l_e = -\sum_{F}^{E} W f.$$

Hence we find by substitution

$$S = \left[\frac{1}{l_b} \sum_{E}^{D} W b - \frac{l_d}{l_g} \sum_{D}^{G} W g - \frac{l_a}{l_e} \sum_{F}^{E} W f \right] - \sum_{E}^{C} W.$$

It is known that the shear is greatest just at the left of some wheel.

In order to compare the shears under (*i. e.*, just at the left of) two wheels at a distance x from each other, let C' and C'' denote their respective positions of the two wheels on the load line. Then using notation previously employed, we find

$$\begin{aligned} S'' - S' = &\left[\sum_{E'}^{D'} + \sum_{D'}^{D''} - \sum_{E'}^{E''} \right] W \frac{b''}{l_b} - \sum_{E'}^{D'} W \frac{(b'' - x)}{l_b} \\ &- \frac{l_d}{l_g} \left[\sum_{D'}^{G'} + \sum_{G'}^{G''} - \sum_{D'}^{D''} \right] W \frac{g''}{l_b} + \frac{l_d}{l_g} \sum_{D'}^{G'} W \frac{(g'' - x)}{l_b} \\ &- \frac{l_a}{l_e} \left[\sum_{F'}^{E'} + \sum_{E'}^{E''} - \sum_{F'}^{F''} \right] W \frac{f''}{l_b} + \frac{l_a}{l_e} \sum_{F'}^{E'} W \frac{(f'' - x)}{l_b} \\ &- \left[\sum_{E'}^{C'} + \sum_{C'}^{C''} - \sum_{E'}^{E''} \right] W + \sum_{E'}^{C} W. \end{aligned}$$

Cancel and notice that $b'' - a'' = l_b$, and the equation may then be written in the form,

$$\begin{aligned} \frac{S'' - S'}{x} = &\frac{1}{l_b} \left[\sum_{E'}^{D'} W + \sum_{D'}^{D''} W \frac{b''}{x} - \sum_{E'}^{E''} W \frac{a''}{x} \right] \\ &- \frac{l_d}{l_b\, l_g} \left[\sum_{D'}^{G'} W + \sum_{G'}^{G''} W \frac{g''}{x} - \sum_{D'}^{D''} W \frac{g''}{x} \right] \\ &- \frac{l_a}{l_b\, l_e} \left[\sum_{F'}^{E'} W + \sum_{E'}^{E''} W \frac{f''}{x} - \sum_{F'}^{F''} W \frac{f''}{x} \right] - \frac{1}{x} \sum_{C'}^{C''} W. \end{aligned}$$

In the first bracket put

$$b'' = d'' - l_d, \text{ and } a'' = e'' + l_a;$$

in the last term of the second put $g'' = d'' + l_g$; and in the middle term of the third put $f'' = e'' - l_e$. Cancel, and the equation becomes

Digitized by Google

$$\frac{S'' - S'}{x_1} = \frac{1}{l_b}\left[\sum_{E'}^{D'} W + \sum_{D'}^{D''} W\frac{d''}{x} - \sum_{E'}^{E''} W\frac{e''}{x}\right]$$

$$- \frac{1}{l_b}\left[\sum_{D'}^{G'} W + \sum_{G'}^{G''} W\frac{g''}{x} - \sum_{D'}^{D''} W\frac{d''}{x}\right]\frac{l_d}{l_g}$$

$$- \frac{1}{l_b}\left[\sum_{F'}^{E'} W + \sum_{E'}^{E''} W\frac{e''}{x} - \sum_{F'}^{F''} W\frac{f''}{x}\right]\frac{l_a}{l_e} - \frac{1}{x}\sum_{C'}^{C''} W \quad . . (2)$$

an equation which may be interpreted as shown in Fig. 3.

Let $u\ u$ be the reaction polygon for the span whose length is x; then the first bracket of the last member in (2) expresses the height of u_d above u_e, the second the height of u_g above u_d, and the third the height of u_e above u_f. It is evident, from similarity of triangles, that the second bracket multiplied by $l_d \div l_g$ expresses the height of u_d above t_b; and the product of the third bracket by $l_a \div l_e$ expresses the height of t_a above u_e: hence the total coefficient of $1 \div l_b$ expresses the height of t_b above t_a. This height divided by l_b is the tangent of the angle which the slope line $t_a\ t_b$ makes with the horizontal.

Again, the coefficient of $1 \div x$ in the last term of (2) is the sum total of the wheel weights which pass C during the movement x, including that standing at C' and excluding that at C'', and this divided by x is the tangent of the angle which $k'\ k''$ makes with the horizontal. When the slope of $t_a\ t_b$ is the greater, then is $S'' > S'$, and *vice versa*.

It will not be difficult to determine the segments of $A\ B$ in which the greatest shears are dominated by the several wheels near the head of the train, for these wheels cause greater shears than those more distant from the head of the train, as appears at once from the general equation for S given above.

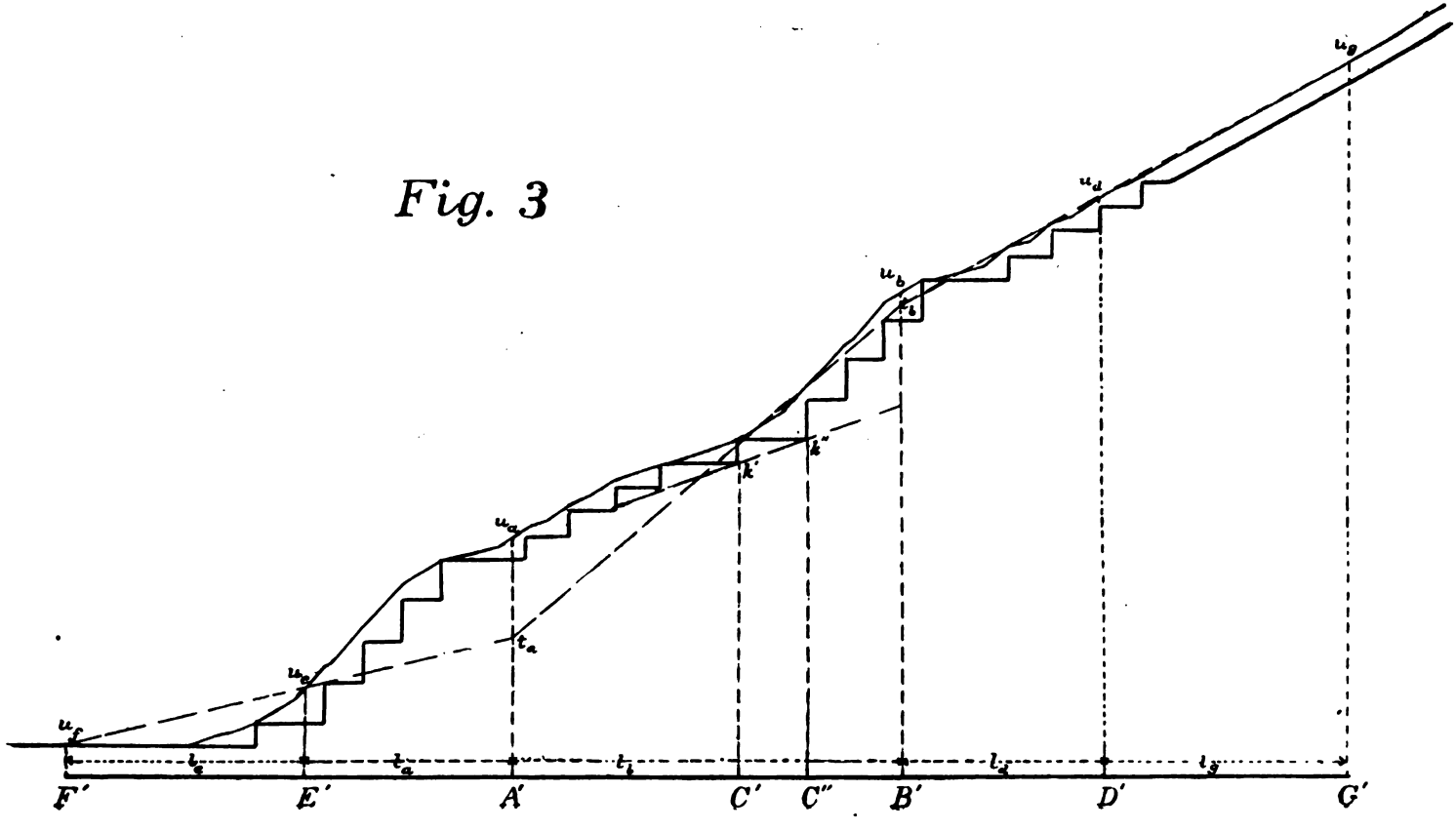
First find the point of $A\ B$ where S_1 the shear just at the left of W_1 is equal to S_2, the shear just at the left of W_2, on the supposition that W_1 does not leave $A\ B$ when W_2 arrives at the point mentioned, as it usually will not. Since the train at most extends from G to some point between A and B, there are no wheels between F and A, so the third bracket vanishes as well as the third term of the first bracket in (2). Hence, in this case, the reaction polygon between u_f and u_a coincides with the horizontal through t_a, as does also the slope line $u_f\ t_a$, as well as the weight line. See Fig. 4.

Now draw $t_a\ t_b$ parallel to $k_1\ k_2$, then the height of t_b above t_a becomes known $= B'\ t_b$.

Now let a card be cut in the shape represented by the broken lines, such that three parallel vertical edges are at the respective distances apart of the verticals at B, D and G. Place the left hand upper corner of the card at the height of t_b above t_a, then shift the card horizontally (so

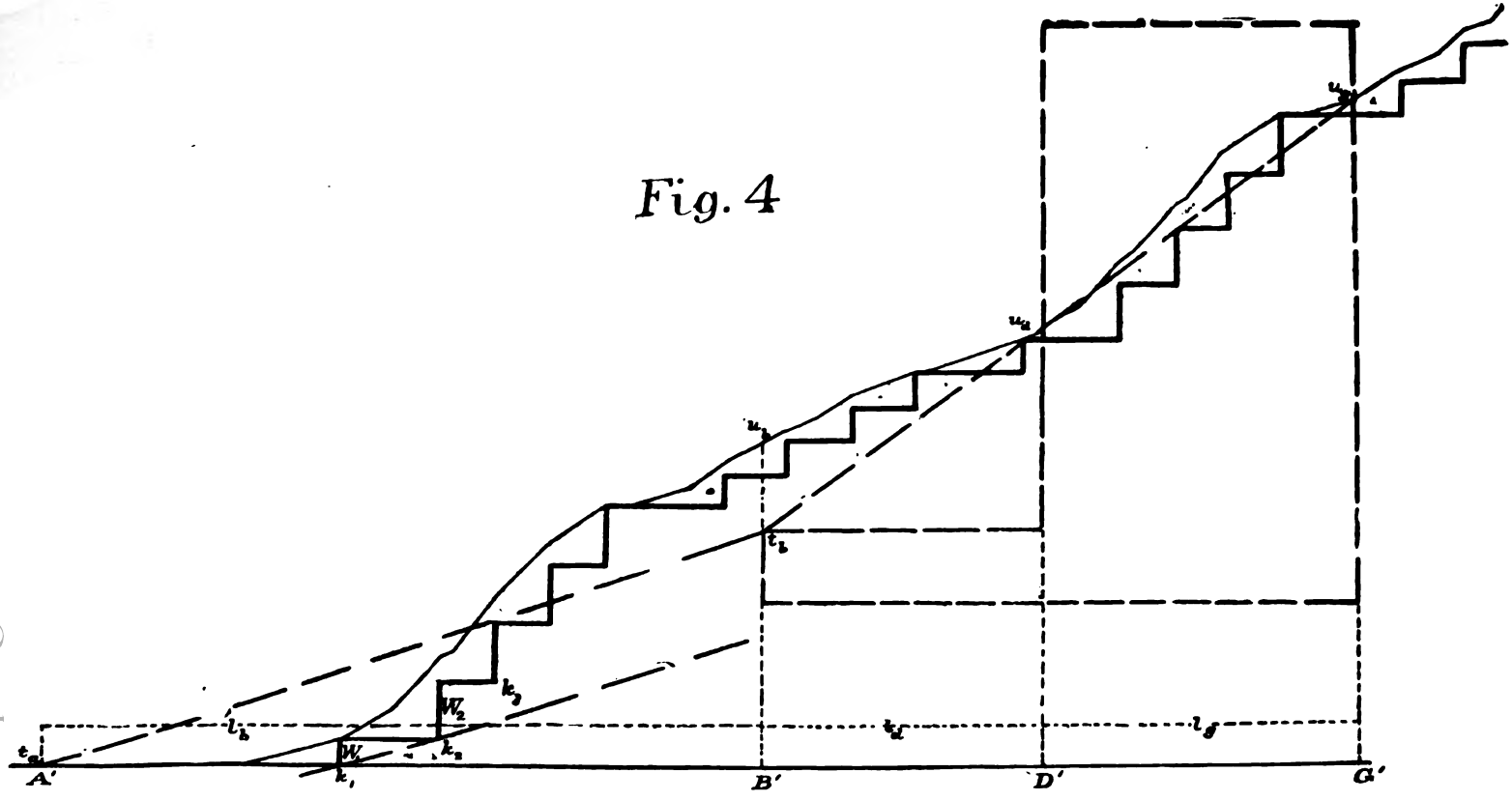
Digitized by Google

Fig. 3



Digitized by Google

Fig. 4



Digitized by Google

that this left upper corner will remain at the same height) until the points u_d and u_g, where the other two vertical edges of the card cut the reaction polygon, fall in the same straight line with the upper left corner. This gives the position of the train, or better, the position of the cantilever when the head of the train has arrived at a point where the shear just at the left of W_1 is the same as the shear just at the left of W_2 will be, when W_2 arrives at that point. The greatest shear in that segment of $A\ B$ at the right of the point so determined is dominated by W_1; and at the left of this point the shear is greatest just at the left of W_2 until W_2 arrives at the point where the shear just at the left of W_2 is equal to that which will exist just at the left of W_2 when W_3 shall arrive at the same point. The construction for finding this second point will be similar to that for the first point, but in this case $t_a\ t_b$ must be drawn parallel to $k_2\ k_3$ instead of $k_1\ k_2$. It will seldom be necessary to find this second point, as the whole of $A\ B$ will be practically dominated either by W_1 or W_2.

Moreover, from the general expression for S given above it not only appears that the shear is greatest near the head of the train, *i. e.*, when the train extends not far to the left of C, but it also appears clear that the shear again begins to increase after the head of the train passes to the left of A, as is evident from the fact that $b > l_b$ for those wheels at the left of A. And since f is really negative in $F\ E$, the shear at C will generally increase until the head of the train reaches F.

A comparison by the method of Fig. 3 will show whether the shear at C is greatest when the head of the train is near C or near F.

The general expression for S also shows that the shear is greater, other things being equal, in case the train is so short that no wheels stand between B and G. The length of the wheel base for the locomotive may be such that this condition cannot be exactly fulfilled. Nevertheless, it should be assumed that the train consists of one locomotive and tender, or two locomotives and tenders with or without uniform train, according as the one or the other assumed loading will cause the greater shear at C. The assumed uniform train should be taken as not extending to the right of B in case that assumption tends to increase the shear.

If no loading is regarded as resting upon that part of the cantilever at the right of B, and it is desired to compare by the method of Fig. 3, the shear at C in case the head of the train is near C with the shear at C in case the head of the train is near F, then it will be necessary to consider B as a free end supported on a pier at B without constraint, or what is the same thing, take l_d and l_g as of infinitesimal length, so that D and G may be for the moment regarded as coinciding with B, though in reality they do not so coincide. This will make t_b coincide with u_b in Fig. 3.

It is to be noticed that in case the greatest shear occurs with the head

Digitized by Google

of the train near C, and there is no load to the right of B, this differs not at all from the case of the greatest shear at C in a simple span $A\ B$, as given in the previous paper. Having now discussed briefly the greatest shear in a simple girder cantilever, we shall proceed to treat the single intersection truss cantilever.

4. To discover the position of the cantilever $E\ D$, Fig. 1, with reference to the train when the shear is greatest in the web members of a given panel $H\ K$, in case the cantilever is a single intersection truss with parallel chords.

1°. In the end span $B\ D$ at any point C of the panel $H\ K$, the shear in the truss may be written—

$$S = \sum_{D}^{G} W \frac{g}{l_d} + \sum_{C}^{D} W - \sum_{H}^{K} W \frac{k}{p} + \sum_{H}^{C} W$$

in which equation p is the length of the panel $H\ K$, and k is the distance of any wheel to the left of its right extremity.

Let the train be supposed to move an indefinitely small distance, $\delta\ x$ toward the left, then—

$$S + \delta\ S = \sum_{D}^{G} W \frac{(g + \delta\ x)}{l_d} + \sum_{H}^{D} W - \sum_{H}^{K} W \frac{(k + \delta\ x)}{p}$$

Hence

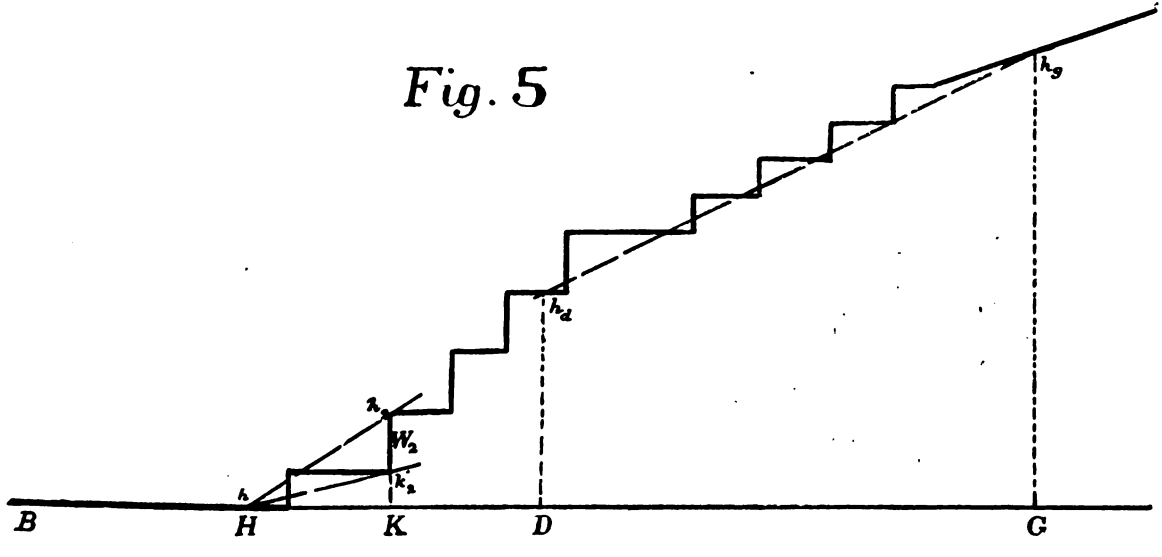
$$\frac{\delta\ S}{\delta\ x} = \frac{1}{l_d} \sum_{D}^{G} W - \frac{1}{p} \sum_{H}^{K} W \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (3)$$

From equation (3) it appears that when the head of the train, as it moves from G toward B, has entered upon the panel $H\ K$ so far that some one of the wheels has arrived at K, which changes the sign of the last member of (3) from positive to negative as it passes K, then will S reach a maximum which is also its numerically greatest value. The first term in the last member of (3) is the tangent of the slope of the line $h_d\ h_g$ in Fig. 5, while the last term is that of the slope line in $H\ K$, *i. e.*, either $h\ h_2$ or $h\ k_2$, according as W_2 stands or does not stand in the panel. Hence it is evident that maximum value of S occurs in $H\ K$ when W_2 stands at K. The general slope of $h_d\ h_g$ will not greatly vary during the passage of the train; hence the same wheel will usually command the greatest shear in every panel of $B\ D$.

2°. In the middle span $A\ B$, at any point C of the panel $H\ K$, the shear in the truss may be written as the difference between the shear in the cantilver regarded as a girder and the shear borne by the stringer

Digitized by Google

Fig. 5



Digitized by Google

$H K$, compare Art. 3. Hence—

$$S = \frac{1}{l_b}\left[\sum_{E}^{D} W b - \frac{l_d}{l_g}\sum_{D}^{G} W g - \frac{l_a}{l_e}\sum_{F}^{E} W f\right] - \sum_{E}^{C} W$$

$$- \frac{1}{p}\sum_{H}^{K} W k + \sum_{H}^{C} W,$$

in which equation k is the distance of any wheel to the left of the right panel point K.

Now, if the train move an indefinitely small distance δx toward the left, we obtain in the same manner as before—

$$\frac{\delta S}{\delta x} = \frac{1}{l_b}\left[\sum_{E}^{D} W - \frac{l_d}{l_g}\sum_{D}^{G} W - \frac{l_a}{l_e}\sum_{F}^{E} W\right] - \frac{1}{p}\sum_{H}^{K} W. \quad \ldots\ldots (4)$$

The terms in this equation have interpretations like those in equation (2), except that the last term is the tangent of the angle which the slope line in the panel $H K$ makes with the horizontal. For a maximum value of S, the last member of (4) must change from positive to negative, which change will evidently occur just as some wheel enters the panel at K.

Equation (4) is somewhat more simple than equation (2) because the reaction polygon in this case is simply the weight line, *i. e.*, the reaction polygon for zero span. It will be convenient to go on and determine the segments of $A B$, in which the maximum shear is dominated by the successive wheels near the head of the train, as may be readily done by the method of Fig. 4, provided the points u_b u_d u_g be taken upon the zero reaction polygon, *i. e.*, upon the weight line, and provided the slope line of the panel having the inclination $W_1 \div p$ be taken instead of k_1 k_2, and the slope line of the panel having the inclination $(W_1 + W_2) \div p$ be taken instead of k_2 k_3, etc., etc. It is also to be noticed in this connection that the general expression for S which has been employed in obtaining equation (4) renders it evident that, other things being equal, the shear at C will be greater in case no part of the train stands between B and G, for b is negative for wheels at the right of B. Hence the train should be taken to consist of one locomotive and tender, or two locomotives and tenders followed by a uniform train or not, according as the one or the other supposition will cause the greater shear at C. The assumed uniform train should not in any case extend to the right of B.

In case there is no load at the right of B and the head of the train is near C, the construction for the greatest shear will be identical with that given in Part I, for the greatest shear in a single intersection truss of span $A B$.

Furthermore, it is evident from the general value of S just referred

Digitized by Google

to, that one maximum value of the shear occurs when the head of the train is near C, and another when it is near F, since f is negative for wheels standing to the right of F.

The concluding remarks of the last article respecting the comparison of the maximum shear at C in case the head of the train is near C, with the maximum shear at C in case the head of the train is near F, apply equally to the problem at present under discussion.

5. To discover what position of the train produces the greatest negative bending moment at any point C in the end span $B\,D$ of the cantilever $E\,D$. Figs. 1 and 6.

Let the sign of the bending moment as usual be taken as positive when produced by forces acting upward, then the moment M at C may be written:

$$M = -\,V_d\,d_c + \sum_{C}^{D} Wc$$

in which equation d_c denotes the distance from D to C; and c as before stated is the distance from C to any wheel, and is consequently negative for every wheel to the right of C.

But $V_d\,l_g = \sum_{D}^{G} Wg$, hence substituting for V_d we have

$$M = -\,\frac{d_c}{l_g}\sum_{D}^{G} Wg + \sum_{C}^{D} Wc$$

Now, let the train move an indefinitely small distance $\delta\,x$ toward the left and we find in the same manner as before:

$$\frac{\delta\,M}{\delta\,x} = -\left[\frac{d_c}{l_g}\,L_g - D_c\right] \qquad (5)$$

in which equation D_c denotes the load standing upon the segment $D\,C$. In order that M may reach a maximum negative value the parenthesis in (5) must change from positive to negative, *i. e.*, the last member of (5) must change from negative to positive. This change can occur in one way only, viz., some wheel may enter upon $C\,D$ at D, which will effect the change, for this will decrease L_g and increase D_c by an amount equal to the weight of the wheel which enters at D. And this is the only way that such a change can possibly be effected, for it is evident that when a wheel enters $D\,G$ at G, the effect is simply to increase L_g, and when a wheel leaves $C\,D$ at C the effect is to decrease D_c, but either of these changes tend to make the parenthesis of equation (5) positive and not negative. The graphical method of discovering which wheel must stand at D in order that M may reach a negative maximum at C is evidently identical with that employed in Part I, for discovering the position of the train which produces a maxi-

Digitized by Google

mum at D in a simple girder of span $C\,G$. The truth of this statement may be made evident as follows:

Without actually drawing any figure, but using the notation previously employed, let h_c and h_g be the points of the weight line at the verticals of C and G, respectively. Then equation (5) requires the two slope lines $h_c\,h_d$ and $h_d\,h_g$ to both lie on one side of $h_c\,h_g$, and the lines $h_c\,k_d$ and $k_d\,h_g$ to lie on the other side of $h_c\,h_g$. The construction may also be arranged thus: Prolong the slope lines $h_g\,h_d$ and $h_g\,k_d$ until they cut the vertical at C, then must h_c lie between these lines so prolonged if M is to reach a negative maximum.

Now, there may be several different positions of the train, which make the negative moment at C a maximum.

To find which bending moment is numerically greatest it will be sufficient to be able to effect a comparison of the bending moments at C for any two different positions of the cantilever with respect to the train.

Denote the respective positions of $C\,D\,G$, distant from each other by x, as before by $C'\,D'\,G'$ and $C''\,D''\,G''$, and using the value of M, already given in this article, we find:

$$M'' - M' = -\frac{d_c}{l_g}\left[\sum_{D'}^{G'} + \sum_{G'}^{G''} - \sum_{D'}^{D''}\right] W g'' + \frac{d_c}{l_g}\sum_{D'}^{G'} W(g'' - x)$$

$$+\left[\sum_{C'}^{D''} + \sum_{D'}^{D''} - \sum_{C'}^{C''}\right] W c'' - \sum_{C'}^{D'} W(c'' - x),$$

in which equation we have put $g' = g'' - x$, and $c' = c'' - x$.

Cancelling and then substituting $g'' = l + d''$ in the last term of the first bracket, and $c'' = d'' - d_c$ in the second term of the second bracket, and then cancelling again, we have finally

$$\frac{M'' - M'}{x\,d_c} = -\frac{1}{l_g}\left[\sum_{D'}^{G'} W + \sum_{G'}^{G''} W\,\frac{g''}{x} - \sum_{D'}^{D''} W\,\frac{d''}{x}\right]$$

$$+\frac{1}{d_c}\left[\sum_{C'}^{D'} W + \sum_{D'}^{D''} W\,\frac{d''}{x} - \sum_{C'}^{C''} W\,\frac{c''}{x}\right] \ldots\ldots\ldots (6)$$

It will be unnecessary to draw a figure here to give the graphical significance of equation (6), but using the notation before employed, let $u_c\,u_d\,u_g$ be points on the reaction polygon for the span x, at the verticals through $C'\,D'\,G'$ respectively. Equation (6) shows that in case the slope of the line $u_d\,u_g$ is steeper than that of $u_c\,u_d$, then is $M'' < M'$, and *vice versa*. But when $u_d\,u_g$ is the steeper, then is u_d below the slope line $u_c\,u_g$, and so $M'' < M'$.

Digitized by Google

Hence the construction is the same as that employed in Part I, for the case of the greater moment at D in a simple girder $C\ G$, except that in the present case the sign is reversed, *i. e.*, the smaller moment has the greater negative value, so that the moment which has actually the greatest numerical value is determined precisely as in the simple girder given in Part I.

The method which has now been given for discovering the position for the numerically greatest bending moment in the cantilever $E\ D$ suffices to find the position for the greatest stress in the flanges of a girder, or in the bars of the chord of a single intersection truss opposite the loaded pin points, or in the bars of both chords of a single intersection truss in the common case of intermediate suspenders or supports.

6. To discover the position of the train which causes the greatest stress in a bar $H\ K$, in the end span $B\ D$, of a single intersection cantilever truss $E\ D$, when the bar $H\ K$ is a member of the chord opposite C, a pin point of the unloaded chord.

In order to obtain the bending moment M, sustained by the truss itself, it will be necessary to subtract from the bending moment obtained in Art. 5, the bending moment in the panel $H\ K$. Since that moment was negative the result so obtained will be numerically the sum of these moments, and may be written :

$$M = -\frac{d_c}{l_g}\sum_{D}^{G} Wg + \sum_{C}^{D} Wc - \frac{c_h}{p}\sum_{H}^{K} Wk + \sum_{H}^{C} Wc,$$

in which equation p denotes the length of the panel $H\ K$, and c_h denotes the distance which H is to the left of C. Let the train move a small distance $\delta\ x$ toward the left, and we then obtain, by an algebraic process like that already employed, the expression :

$$\frac{\delta\ M}{\delta\ x} = -\left[\frac{d_c}{l_g}\sum_{D}^{G} W - \sum_{H}^{D} W + \frac{c_h}{p}\sum_{H}^{K} W\right] \ldots\ldots\ldots (7)$$

The meaning of this equation can be readily made evident by a figure similar to those already given. A rough diagram, such as the reader will at once supply from the following description, will make the interpretation clear. Let H be the pin point at the left end and K that at the right end of the panel $H\ K$ in the end span $B\ D$.

Call the points where the verticals at H, K, D and G cut the weight line h_h, h_k, h_d and h_g respectively. Draw the slope line $h_h\ h_k$ for the panel and call the point where the vertical at C cuts it j_c. Draw also the slope line $h_d\ h_g$ and prolong it until it intersects the vertical at C at the point t_c.

Digitized by Google

Now the last term in equation (7) expresses the height of j_c above h_k; the middle term is the height of h_d above h_k, and the difference of the two thus found is the difference in height between j_c and h_d. But the first term is the difference in height between t_c and h_d, and the entire quantity within the parenthesis in equation (7) is therefore merely an expression for the height $j_c\, t_c$, *i. e.*,

$$\frac{\delta\ M}{\delta\ x} = -\ j_c\ t_c,$$

in which equation $j_c\ t_c$ is positive in case t_c is above j_c and negative in case it is below.

But without drawing $h_d\ h_g$ or locating the point t_c at all, it is evident that $j_c\ t_c$ is positive or negative according as h_d lies above or below the slope line $j_c\ h_g$, as shown by a stretched thread.

In order that M may reach a negative maximum the quantity in the parenthesis in equation (7) must change from positive to negative, which change can occur at the instant when some wheel, say $W_d = h_d\ k_d$ reaches D, such that the slope line $j_c\ h_g$ cuts the weight line between h_d and k_d; but this change cannot occur when any wheel comes on at G, for that only increases the quantity in the parenthesis; nor can it occur when any wheel leaves the panel at H, for that has more effect to diminish numerically the negative term of the parenthesis than to diminish the positive term. But these are the only changes taken account of in equation (7), consequently the negative maximum, *i. e.*, the maximum numerical value of M, is reached only when some wheel reaches D.

In order to compare any two bending moments, negative maxima or not, at points of the train distant x from each other, denote the left hand position of the points $H\ C\ K$, etc., in the span $B\ D$ by $H'\ C'\ K'$, etc., and the right hand position of $H''\ C''\ K''$, etc. Then if we compare the expression for M in the first part of this article with that in the previous article, it appears that they differ in this, that the value of M we are now treating has in it a pair of terms due to the panel $H\ K$ and point C, and of the same signs as those due to the span $D\ G$ and point C. If we then compare the moments at C' and C'' in the panel $H\ K$, in a manner like that by which we obtained equation (6), we shall obtain an equation having in it all the terms found in equation (6), together with additional terms for the panel $H\ K$, which can be obtained as they were in the corresponding investigation in Part I, but which we can readily write by symmetry. We may therefore venture to write out the equation without further preliminaries, as follows:

Digitized by Google

$$\frac{M'' - M'}{x} = -\frac{d_c}{l_g}\left[\sum_{D'}^{G'} W + \sum_{G'}^{G''} W\frac{g''}{x} - \sum_{D'}^{D''} W\frac{d''}{x}\right] + \left[\sum_{C'}^{D'} W + \sum_{D'}^{D''} W\frac{d''}{x} - \sum_{C'}^{C''} W\frac{c''}{x}\right] - \frac{c_h}{p}\left[\sum_{H'}^{K'} W + \sum_{K'}^{K''} W\frac{k''}{x} - \sum_{H'}^{H''} W\frac{h''}{x}\right] + \left[\sum_{H'}^{C'} W + \sum_{C'}^{C''} W\frac{c''}{x} - \sum_{H'}^{H''} W\frac{h''}{x}\right] \quad \ldots\ldots (8)$$

Using the notation previously employed, the reader will be able by help of the description which follows to supply the figure needed in the interpretion of equation (8). Let u_c, u_d, u_g, etc., be the points on the reaction polygon x at the verticals through C', D', G', etc. Then is the quantity in the first parenthesis the height of u_g above u_d. Let the slope line $u_d\ u_g$ cut the vertical at C' in t_c. The first group term is the difference in height between t_c and u_d, and the first two terms taken together are equal to $u_c\ t_c$, *i. e.*, the height of t_c above the reaction polygon at the vertical through C', positive when t_c is above u_c, and negative when below.

The quantity in the third parenthesis is the height of u_k above u_h; and if the slope line $u_h\ u_k$ be drawn and we mark the point i_c where it cuts the vertical through C, then is the third group term the expression for the difference in height between u_h and i_c; and the fourth parenthesis is the height of u_c above u_h. Hence the last two terms together denote the height $u_c\ i_c$, negative in case i_c is above u_c and positive in case it is below. Hence, taken all together, the last member is the difference in height between i_c and t_c, *i. e.*,

$$\frac{M'' - M'}{x} = i_c\ t_c,$$

in which $i_c\ t_c$ is positive in case t_c is above i_c but negative in case it is below i_c.

Now, both M'' and M' are intrinsically negative, hence M'' is numerically greater than M' in case $i_c\ t_c$ is negative, *i. e.*, in case t_c falls below i_c.

It will be more convenient to stretch a thread so as to form the slope line $i_c\ u_g$, and not draw $u_g\ t_c$ at all; then in case u_d falls below $i_c\ u_g$ we have also t_c below i_c and *vice versa*. Hence, in case u_d falls below $i_c\ u_g$, M'' is numerically greater than M'; but when it is above, M' is numerically the greater.

It will be noticed that the bending moment and shear at a point C in the end span $B\ D$ depend in no way upon any loading except that standing between C and G in case of a girder, and upon that between H and K in addition, in case of a single intersection truss. The constructions

Digitized by Google

which have been given will, therefore, answer equally well for any point C in the end span $E\,A$, in which the greatest moment and shear will depend only upon the loading between C and F, or C and F, and H and K when $H\,C\,K$ lie in $E\,A$, while the load is still supposed to come on from G.

If the end spans $E\,A$ and $B\,D$ or $D\,G$ and $F\,E$ are not equal in length, it will be necessary first to put one end of the cantilever toward G, and then the other, which is equivalent to supposing the train first to come on at G and then at F.

It will also be best to compare numerically the greatest value of the moment in each end span when the train comes on at G with that when it comes on at F, in order to select the greater.

7. To discover the positions of the train which produce respectively the greatest and least bending moments, positive or negative, at any point C in the middle span $A\,B$ of the cantilever $E\,D$. By taking moments about C of the applied forces at the left C, the value of the bending moment M is found to be $M = V_a\,c_a - V_e\,(c_a + l_a) - \sum_{E}^{C} W c$.

Now, substituting the values of V_a and V_e, given in Art. 3, we have,

$$M = \frac{c_a}{l_b}\left[\sum_{E}^{D} W b - \frac{l_1}{l_g}\sum_{D}^{G} W g - \frac{l_a}{l_e}\sum_{F}^{E} W f\right] + \frac{l_a}{l_e}\sum_{F}^{E} W f - \sum_{E}^{C} W c.$$

Now, let the train move an indefinitely small distance $\delta\,x$ toward the left, and we obtain by a method similar to that before employed:

$$\frac{\delta\,M}{\delta\,x} = \frac{c_a}{l_b}\left[\sum_{E}^{D} W - \frac{l_d}{l_g}\sum_{D}^{G} W - \frac{l_a}{l_c}\sum_{F}^{E} W\right] - \left[\sum_{E}^{C} W - \frac{l_a}{l_e}\sum_{F}^{E} W\right] \quad (9)$$

The second term within the first parenthesis of equation (9), is expressed in Fig. 6 by $m_a\,t_a$, and the third term by $m_b\,t_b$ as appears from similarity of triangles, hence the entire quantity within the parenthesis expresses the height of t_b above t_a. Similarly the last parenthesis of (9) expresses the height above t_a of k_c or of h_c according as W_c ($= h_c\,k_c$) stands at the right or left of C. In case the slope line $t_a\,t_b$ cuts W_c between h_c and k_c, the value of M reaches a maximum at C, for it causes the last member of (9) to change from positive to negative. It will then be necessary to discover by trial which wheel weights on reaching C are cut by $t_a\,t_b$, and this will not be a thing difficult to do in a manner similar to the procedure in previous cases.

The value of M may also reach a maximum either when some wheel enters $D\,G$ at G, or when some wheel leaves $F\,E$ at F. For when a wheel enters at G that suddenly increases the slope of $h_d\,h_g$ and so decreases that of $t_a\,t_b$ without changing any other slope lines; and when a wheel leaves $F\,E$ at F, that decreases the slope of $h_f\,h_e$ and so increases the slope of both $t_a\,t_b$ and $t_a\,h_c$, while no other slope line is altered, but

Digitized by Google

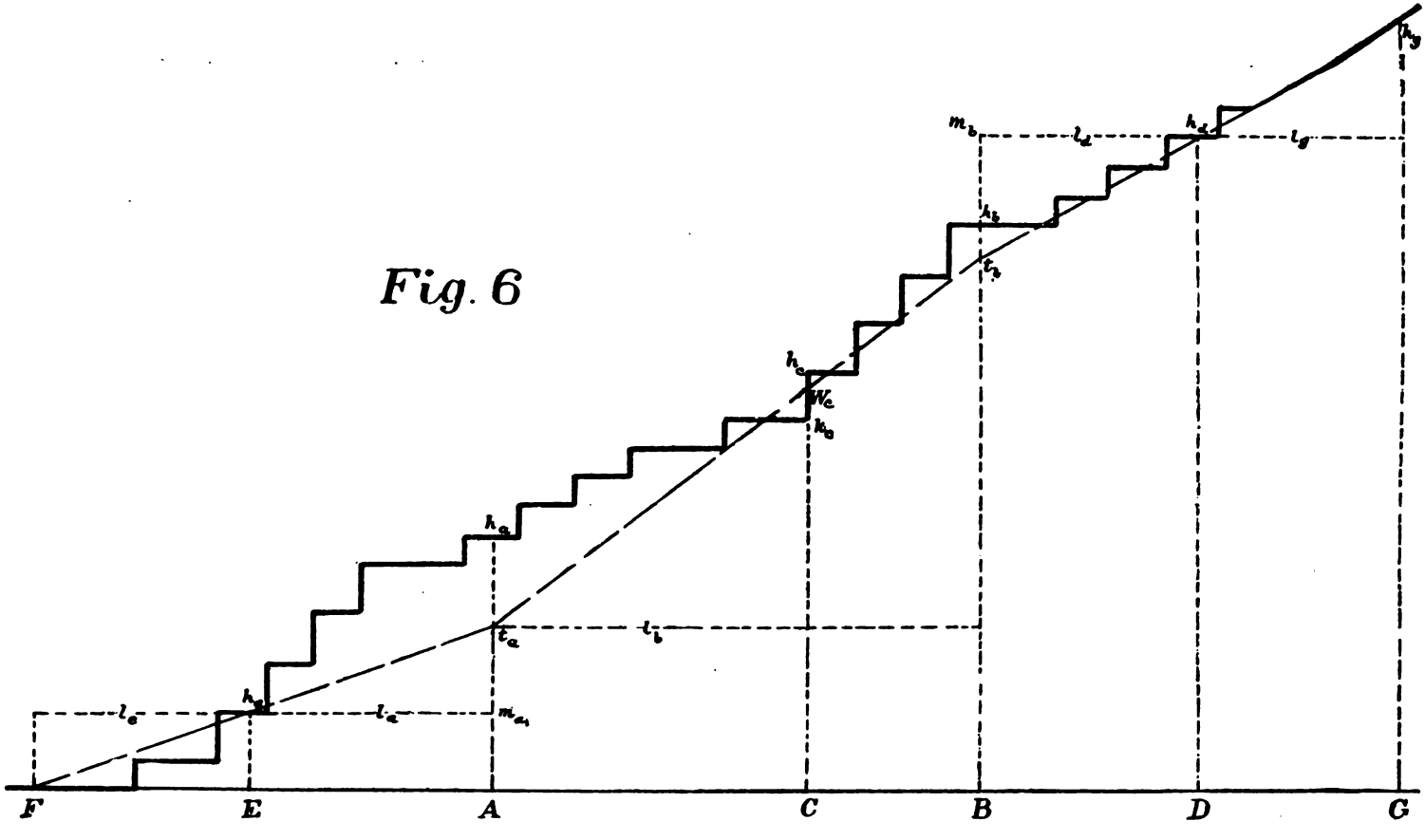


Fig. 6

Digitized by Google

the slope of $t_a\ h_c$ will be increased much more than $t_a\ t_b$. Hence the last member of equation (9) may change from positive to negative, and so make M a maximum either just as a wheel comes on at G or goes off at F. The same result follows from considering the change in the numerical value of the terms which constitute the last member, equation (9). Again, in order to find a position of the train which will cause M to reach a minimum at C, it is necessary to find when the last member of (9) changes sign from negative to positive. This change can occur either when a wheel leaves $D\ G$ and enters upon $E\ D$ at D, or when it leaves $E\ D$ and enters upon $F\ E$ at E; for these two changes are exactly the reverse of those two just discussed, and they both of them evidently tend to make the last member of equation (9) positive. The same results appear from a consideration of the changes in the numerical value of the terms constituting the last member of equation (9).

In case the cantilever is a single intersection truss, and not a solid girder just as in the case previously treated, those positions of the train which cause M, the bending moment borne by the truss, to reach a maximum or a minimum value at a pin point C of the unloaded chord, *i. e.*, cause the maximum or minimum stress in a bar $H\ K$ of the loaded chord, are to be found by using the point j_c at the intersection of the slope line $h_h\ h_k$ with the vertical at C instead of the point h_c in the weight line. It will be unnecessary here to draw the figure, which will be readily supplied by the reader who examines Fig. 10, Part I, and equation (7) Part I; from which it appears that we can obtain the equation needed to treat this case by simply annexing to equation (9) Part II the terms relating to the panel $H\ K$, viz.:

$$-\left[\frac{c_h}{p}\sum_{H}^{K} W-\sum_{H}^{C} W\right]$$

It then appears just as shown in Part I, that the point j_c may be suddenly lifted upwards either when a wheel enters the panel $H\ K$ at K, or when a wheel leaves it at H in such a manner as to pass the point i_c situated upon $t_a\ t_b$. Hence the stress in $H\ K$ may reach a maximum value when some wheel stands at either of the four points H, K, F or G, and a minimum at either of the two points E or D. Which wheel of the train produces a maximum or a minimum at C must be discovered by trial by methods of procedure like those already employed.

It is to be noticed, however, in this connection, that in order to obtain the most unfavorable case on the whole, it will be necessary to consider the general value of M which precedes equation (9). From that equation it is evident that in order for M to be as great as possible, there should be no loading upon any part of $F\ G$ except upon $A\ B$ alone, and the train in the most unfavorable case should approximate as near this as the actual construction of locomotives and tenders will practically permit. In case it is possible to have no load except that

Digitized by Google

upon $A\,B$, the maximum moment due to the moving load is the same as that in the simple span $A\,B$.

Again, in order that the value of M may be the least possible at any point of $A\,B$ it appears from the same equation that there should be no loading upon $A\,B$. The most unfavorable case possible will then be assumed by supposing the rest of $F\,G$ to be covered by locomotives with tenders and trains.

In treating this last combination of two assumed trains it will be best to find the train position for a minimum at C due to a train approaching from G, when its first wheel is near B, and then a train position for a minimum at C due to a train approaching from F when its first wheel is near A. This combination will usually produce the most unfavorable case of a minimum or negative maximum at C.

8. In order to discover which of two positions of the train causes the greater bending moment at any point C of the span $A\,B$, the following comparison is given of the two values of M at any distance x from each other along the train.

The steps of the analysis are strictly analogous to those before employed, and need not be given in quite such detail as in the previous demonstrations. Employing the value of M, from which equation (9) is derived, we have:

$$\begin{aligned}
M''-M' = {} & \frac{c_a}{l_b}\left[\sum_{E'}^{D'} + \sum_{D'}^{D''} - \sum_{E'}^{E''}\right] W\,b'' - \frac{c_a}{l_b}\sum_{E'}^{D'} W\,(b''-x) \\
& - \frac{c_a\,l_d}{l_b\,l_g}\left[\sum_{D'}^{G'} + \sum_{G'}^{G''} - \sum_{D'}^{D''}\right] W\,g'' + \frac{c_a\,l_d}{l_b\,l_g}\sum_{D'}^{G'} W\,(g''-x) \\
& - \frac{c_b\,l_a}{l_b\,l_e}\left[\sum_{F'}^{E'} + \sum_{E'}^{E''} - \sum_{F'}^{F''}\right] W\,f'' + \frac{c_b\,l_a}{l_b\,l_e}\sum_{F'}^{E'} W\,(f''-x) \\
& - \left[\sum_{E'}^{C'} + \sum_{C'}^{C''} - \sum_{E'}^{E''}\right] W\,c'' + \sum_{E'}^{C'} W\,(c''-x)
\end{aligned}$$

in which equation $c_b = c_a - l_b$. Hence

$$\begin{aligned}
\frac{M''-M'}{x} = {} & \frac{c_a}{l_b}\left[\sum_{E'}^{D'} W + \sum_{D'}^{D''} W\,\frac{(d''-l_a)}{x} - \sum_{E'}^{E''} W\,\frac{(l_a+l_b+e'')}{x}\right] \\
& - \frac{c_a\,l_d}{l_b\,l_g}\left[\sum_{D'}^{G'} W + \sum_{G'}^{G''} W\,g'' - \sum_{D'}^{D''} W\,\frac{(d''+l_g)}{x}\right] \\
& - \frac{c_b\,l_a}{l_b\,l_e}\left[\sum_{F'}^{E'} W + \sum_{k'}^{E''} W\,\frac{(e''-l_e)}{x} - \sum_{F'}^{F''} W\,\frac{f''}{x}\right] \\
& - \left[\sum_{E'}^{C'} W + \sum_{C'}^{C''} W\,\frac{c''}{x} - \sum_{E'}^{E''} W\,\frac{(c_a+l_a+e'')}{x}\right]
\end{aligned}$$

Digitized by Google

$$\therefore \frac{M''-M'}{x} = \frac{c_a}{l_b}\left[\sum_{E'}^{D'} W + \sum_{D'}^{D''} W\frac{a''}{x} - \sum_{E'}^{E''} W\frac{e''}{x} - \frac{l_d}{l_g}\left(\sum_{D'}^{G'} W + \sum_{G'}^{G''} W\frac{g''}{x} - \sum_{D'}^{D''} W\frac{d''}{x}\right) - \frac{l_a}{l_e}\left(\sum_{F'}^{E'} W + \sum_{E'}^{E''} W\frac{e''}{x} - \sum_{F'}^{F''} W\frac{f''}{x}\right)\right] - \left[\sum_{K''}^{C'} W + \sum_{C'}^{C''} W\frac{c''}{x} - \sum_{E'}^{E''} W\frac{e''}{x} - \frac{l_a}{l_e}\left(\sum_{F'}^{E'} W + \sum_{E'}^{E''} W\frac{e''}{x} - \sum_{F'}^{F''} W\frac{f''}{x}\right)\right] \quad \ldots (10)$$

It will be unnecessary to do more than to describe the figure which expresses the relations in the last member of (10), as it can be easily supplied by the reader. Let $u_f\ u_e$, etc., be the reaction polygon for the span x. Now prolong the slope line $u_f\ u_e$ until it intersects the vertical through A at t_a. Likewise prolong $u_d\ u_g$ until it intersects the vertical through B at t_b. Let i_c be the point where the line $t_a\ t_b$ intersects the vertical through C. Then is

$$\frac{M''-M'}{x} = i_c\ u_c$$

in which u_c is the point of the reaction polygon at the vertical C. In case u_c is above i_c then $M''<M'$, and *vice versa*.

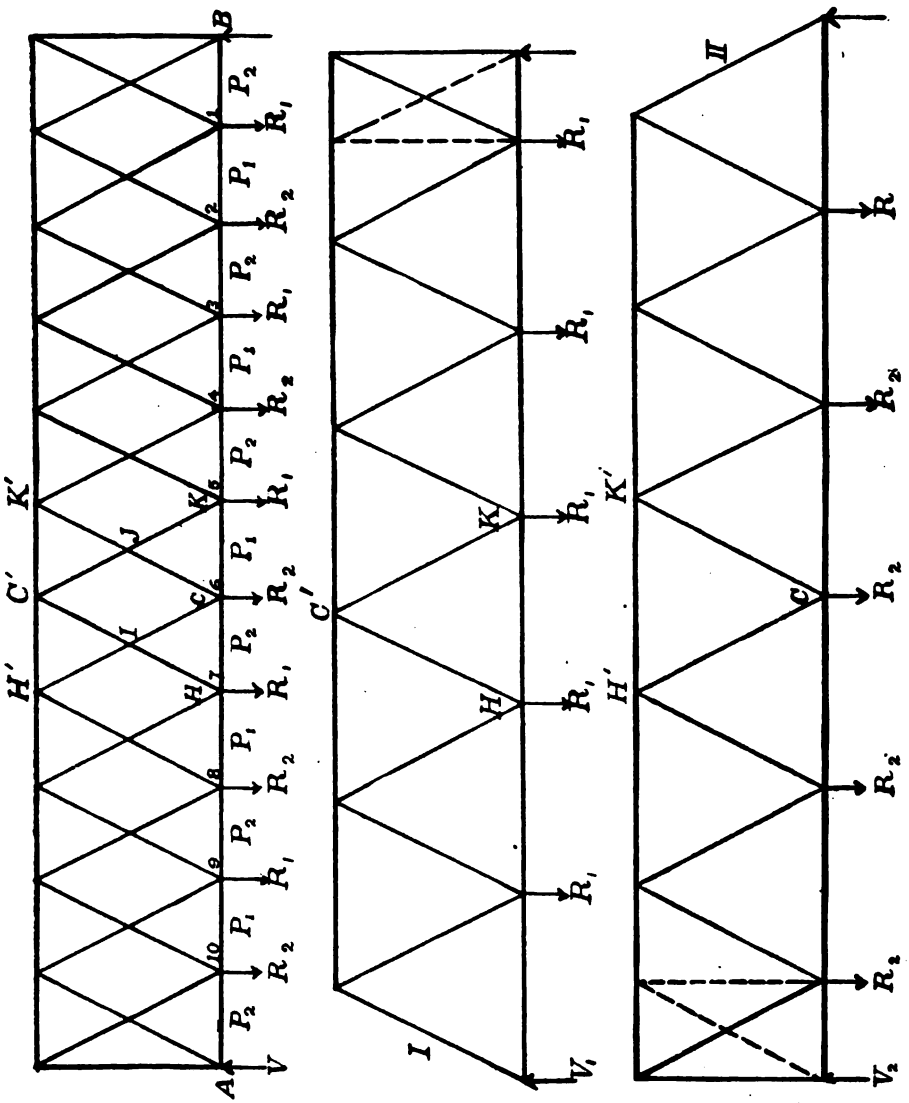
In case it is desired to compare the stresses in a bar HK of the loaded chord of single intersection truss cantilever we must merely employ the point j_c at the intersection of $u_h\ u_k$ and the vertical at C instead of the point u_c, which statement appears sufficiently evident from previous similar discussions.

9. The double intersection truss with end supports.

It is usual to assume that a truss of this kind consists of two components, each of which is a single intersection truss, and that these two components are superposed upon each other in such a manner that while they have their chord members in common, the web members of the two systems are more or less distinct from each other. The mutual independence of the two systems of web members is usually such that it is regarded as sufficiently accurate for practical purposes to assume that any load applied at a panel point of one system causes no stresses in the web members of the other system. This assumption is taken as the basis of the following investigation which has for its object the establishment of graphical or semi-graphical methods, in connection with the reaction polygons due to the wheel weights of the train, for discovering the train

Digitized by Google

Fig. 7



Digitized by Google

position which causes the greatest stress in any member of a given double intersection truss.

10. To discover the train position which produces a maximum stress in a given web member of a double intersection truss having inclined web members as shown in Fig. 7.

This maximum stress occurs, as is well known, coincidently with the greatest shear in that panel of the component truss in which the web member is situated, for the shear is the vertical component of the stress in the member.

In order then to investigate the algebraic condition for a maximum value of the shear in a panel of a component truss, let Fig. 7 represent a double intersection truss and its components I and II; and let it be required to find the condition for a maximum shear in the panel HK of component I, *i. e.*, in the two web members of this panel.

Denote the pin load at any panel point of component I by R_1 and that at any panel point of II by R_2. Also designate by P_1 the load in any panel just at the left of any one of the panel points of I, *e. g.*, just at the left of H or K; and designate by P_2 the load in any panel just at the left of either of the panel points of II, as shown in Fig. 7.

Then any R_1, as for example R_1 at K, is due to P_1 at its left and P_2 at its right; while any R_2, as for example R_2 at C is due to P_2 at its left and P_1 at its right.

Then

$$R_1 = \frac{1}{p}(P_2\, c_2 - P_1\, c_1)$$

Let the panel points in II, which are next left and next right of K in I, be denoted respectively by C_1 and C_2, and let P_1 and P_2 be the panel loads respectively at the left and right of R_1 at K, while c_1 and c_2 denote respectively the distances of the centers of gravity of P_1 and P_2 from C_1 and C_2, these distances being taken as positive toward the left and negative toward the right. We see that c_1 in Fig. 8 is negative.

Now the shear S_1 in any panel HK of I may be written—

$$S_1 = \sum_A^B R_1 \frac{b_1}{l} - \sum_A^C R_1$$

in which equation b_1 denotes the distance of any pin load R_1 from B. Substitute in this equation the value of R_1 just found, and we find—

$$p\, S_1 = \sum_A^B (P_2\, c_2 - P_1\, c_1)\, \frac{b_1}{l} - \sum_A^C (P_2\, c_2 - P_1\, C_1)$$

Now let the train move an infinitesimal distance δx toward the left. In this new position c_2 and c_1 become respectively $c_2 + \delta x$ and $c_1 + \delta x$, and we find after reduction—

Digitized by Google

$$p\frac{\delta S_1}{\delta x} = \sum_{A}^{B}(P_2 - P_1)\frac{b_1}{l} - \sum_{A}^{C}(P_2 - P_1)$$

In order to put this last equation into a form somewhat more symmetrical and convenient for discussion, multiply the last summation by $\frac{b_1 - a_1}{l}$ $(=1)$, separate the summation from A to B into two parts, one part from A to C, and the other from C to B, and then cancel. The result may then be put in the form—

$$\frac{l}{2}\frac{\delta S_1}{\delta x} = -\frac{1}{2}\sum_{A}^{C}(P_2 - P_1)\frac{a_1}{p} + \frac{1}{2}\sum_{C}^{B}(P_2 - P_1)\frac{b_1}{p} \quad \ldots(11)$$

In which equation a_1 is the distance from A to any R_1 and is to be taken with the P_2 and P_1 respectively just at the right and left of this R_1, while b_1 is the similar distance from B; consequently $\frac{b_1}{p}$ is an integer as is $\frac{a_1}{p}$. Every a_1 in (11) is negative and every b_1 is positive.

For brevity write equation (11) thus:

$$\frac{l}{2}\frac{\delta S_1}{\delta x} = -\frac{1}{p}(A_1 - B_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots (11_1)$$

in which $-A_1$ is one-half the summation from A to C, and B_1 is one-half that from C to B.

Now, in order to see how S_1 may reach a maximum it is necessary to trace the variations in the value of the last member of (11) and discover under what circumstances it changes from positive to negative. It is so easy to get a clear idea of the numerical changes in the value of (11) in the simple case of a uniform train of unlimited length coming on at B, and this case enables us to gain such an insight into the usual cases of unequal loading, that we shall consider that special case first.

For brevity, designate the first pin point of the loaded chord in I at the left of B as joint 1; the next pin point of I as joint 2, etc.

Now as the head of a uniform train moves from B toward joint 1, that panel load of equation (11) which first comes into existence is the P_2 next at the right of joint 1. This furnishes a positive term in B_1, so that δS_1 is necessarily positive until the head of the train reaches joint 1. As the head of the train moves on toward the left of joint 1, the next panel weight P_1 comes into existence, and by the time this whole panel is covered by the train the quantity $P_2 - P_1$ for joint 1 vanishes and $\delta S_1 = 0$. Up to this point δS_1 has not at any instant been negative, and so the condition for a maximum value of S_1 has not been satisfied.

If now the train continues to move toward the left, it is evident that a series of statements can be shown to be true respecting joint 2

Digitized by Google

precisely like those already shown to hold respecting joint 1; and that meanwhile that part of B_1 due to joint 1 is zero and remains zero during all the subsequent motion.

When the two panels about joint 2 are both covered, then $P_2 - P_1$ becomes zero for this joint also, and remains so. This state of affairs continually repeats itself at the head of the uniform train until it reaches C, *i. e.*, the value of δS_1 has at no time been negative. Hence S_1 has in general been increasing as the head of the train progressed toward C, though the rate of increase has not been uniform. When now the head of the train arives at C, δS_1 vanishes, but as soon as it has passed beyond C towards A, then a term of the type P_2 in A_1 first comes into existence, and δS_1 then becomes negative for the first time.

The condition for a maximum value of S_1 is thus fulfilled when the head of the uniform train reaches C, and it is not fulfilled again, as δS_1 does not again become positive.

To return now to the more general case where a train of unequal wheel weights enter upon the span at B, we see that the first P_2 comes into existence as the head of the train approaches joint 1, and the first P_1 makes its appearance when the head of the train has passed joint 1. In case $P_2 - P_1$ for joint 1 becomes negative before there is any pin load on joint 2, the condition for a maximum value of S_1 is then fulfilled, but the maximum so reached is not in general so great as that reached when the head of the train is near C. For it is evident, as the train proceeds toward the left and the P_2 for joint 2 comes into existence, that we soon have $\delta S_1 > 0$ for two reasons; first, because the P_2 at joint 2 must in general exceed $P_2 - P_1$ at joint 1; and secondly, the b_1 of joint 2 is greater than the b_1 of joint 1.

It is likewise seen that as the head of the train progresses towards C the value of δS_1 can be negative only occasionally and for short intervals, so that the sign of δS_1 is controlled and kept positive most of the time, just as in case of a uniform train, by the relatively large value of $P_2 b_1$ in the panel near the head of the train. This insures that S_1 shall, with insignificant exceptions, be continually increasing as the train progresses until W_1, the head of the train, is in the neighborhood of C. Consequently the greatest value of S_1 occurs when W_1 is not far from C, a result in accordance with what is well known to be the case for the simple girder and for the single intersection truss.

When any wheel, as for example W_1, stands at C, it evidently tends to make the last member of (11) negative, for in case it is regarded as infinitesimally distant and toward the right of C, it is then part of the P_1 at K, and so enters the second summation of (11) as a negative quantity; but in case it is regarded as standing just at the left of C, it forms part of the P_2 at H, and consequently enters the first summation of (11) as positive, with a negative multiplier a_1. Whether any wheel, as W_1, has a greater influence to render (11) negative, and so produce a

Digitized by Google

maximum of S_1 by standing just at the right or at the left of C, depends upon the relative magnitude of its b_1 and its a_1 in these positions; *i. e.*, in case $A\,H < K\,B$, or what is the same thing, in case $A\,C < C\,B$, then any wheel has more influence to render (11) negative by standing at the right of C than at the left. But when $A\,C > C\,B$ the wheel has more influence standing at the left of C. Hence it follows that a maximum shear may be caused in any panel $H\,K$ in the right half of the span by the passage of W_1, or some other wheel over the panel point C of component II, intermediate between H and K, but the maximum shear cannot be so caused in any panel of the left half of the span.

Hence when there are counterbraces they can reach their greatest stress in this manner only, though they may not reach it in this manner, but instead, at the instant some wheel passes K or some other panel point of component I, between C and B, as will be made to appear later.

Since W_1 cannot be very distant from C when S_1 reaches its greatest value, suppose the train to occupy initially a position such that W_1 is just at the right of C.

We shall now proceed to find out whether the last member of (11) is positive or negative.

Equation (11) is of a form which admits of ready computation, especially since the value of $\frac{1}{2}(P_2 - P_1)$ can be obtained for each R_1 by inspection from the diagram of the weight line as shown in Fig. 8, thus: Mark the panel points of the truss upon the edge of a strip of paper and place it horizontally upon the diagram of the weight line in such a position that C will lie in the vertical at W_1. Let K in Fig. 8 be any panel point of component I, and let C_1 and C_2 be adjacent panel points of the same chord in component II.

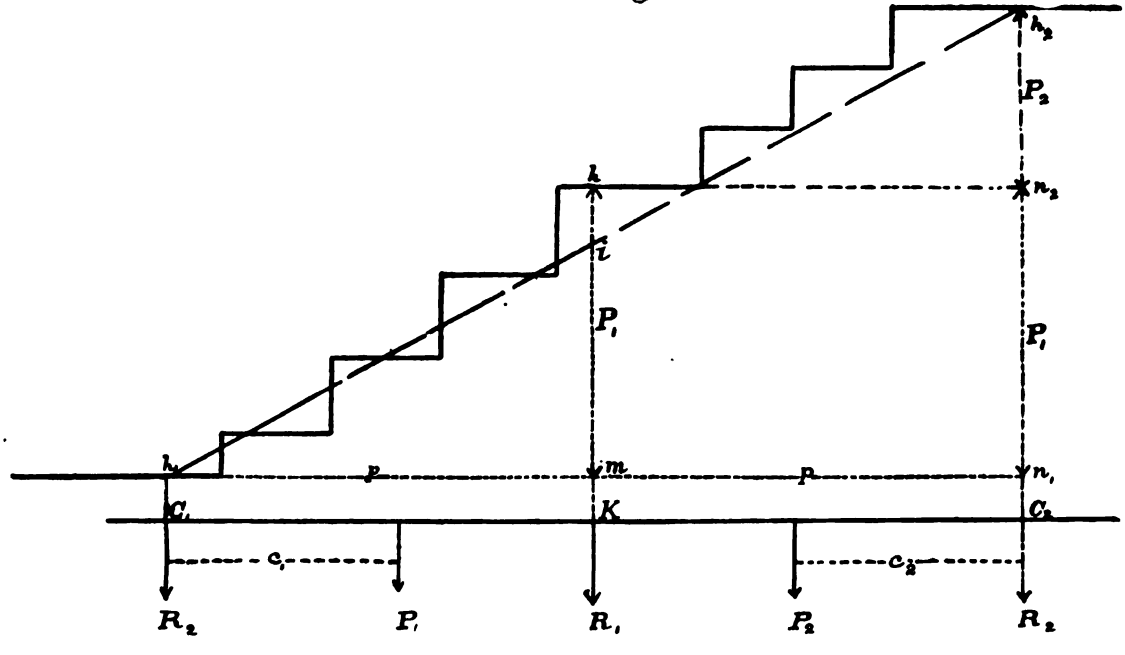
Then is P_1 the height of h above h_1 and P_2 that of h_2 above h. Also $P_2 + P_1$ is the height of h_2 above h_1 and $\frac{1}{2}(P_2 + P_1)$ that of i above h; hence $h\,i = \frac{1}{2}(P_2 + P_1) - P_1 = \frac{1}{2}(P_2 - P_1)$, which quantity is positive in case i is above h, but negative in case i is below h. The value of $\frac{1}{2}(P_2 - P_1)$ at each panel point of component I can thus be read off by scale on the weight diagram without defacing it in any way. Take the numerical value of $\frac{1}{2}(P_2 - P_1)$ thus obtained at each panel point of I, multiply it by $b_1 \div p$, the integer expressing the number of panels the point is from B, and finally take the algebraic sum of these products from C to B.

It will usually be possible to discover whether the last member of (11) is positive by taking the terms at only one or two of the panel points near the head of the train. The result of this brief numerical computation of (11) will show whether $\delta\,S_1$ is positive or negative when W_1 is just at the right of C.

Let us discuss the two cases in which, first, (11) is positive, and, secondly, negative with W_1 just at the right of C. First, in case (11)

Digitized by Google

Fig. 8



Digitized by Google

is positive and C is in the right half of the span, (11) may become negative and make S_1 a maximum, and is likely to do so when W_1 passes C.

The only numerical changes in (11) which will in general occur as W_1 passes from a position just at the right to one just at the left of C, is to increase B_1 by the quantity $W_1 . K B$ and decrease $- A_1$ by $W_1 . A H$, or on the whole decrease (11) by

$$\frac{1}{2} W_1 (A H - K B) = \frac{1}{2} W_1 (A C - C B) = \frac{1}{2} W_1 (2 c_a - l).$$

If the subtraction of this quantity renders (11) negative, then W_1 at C makes S_1 a maximum and in all ordinary cases this is the greatest maximum. If, however, the subtraction of this quantity does not render (11) negative, the train must be moved still further toward the left. During this motion the first change that can occur having a tendency to make (11) negative will take place as soon as either one of the panel points K, etc., of component I in the segment $C B$, or the point C is next passed by any wheel. The next wheel that can pass C is W_2, so that one or other of the points K, etc., between C and B will in general be passed by some wheel before W_2 reaches C. It is to be noticed at the same time that the change in the value of (11) caused by any wheel in passing a panel point of component II between C and B does not tend to make (11) negative but positive.

If in moving the train from its position with W_1 just at the left of C until some wheel, as W_n, passes some point of component I between C and B, as K_n, no wheel has meanwhile passed any point of component II, then the value of (11) will be unchanged until W_n passes K_n.

Whether (11) is unchanged or not, it will be possible to obtain the quantity to be subtracted from (11) when W_n passes K_n as follows. At K_n the P_1 will be increased by W_n, and the P_2 decreased by W_n, consequently $\frac{1}{2} (P_2 - P_1)$ will, on the whole, be decreased by W_n and the quantity to be subtracted from (11) is $W_n \frac{b_1}{p}$, in which expression $b_1 = K_n B$, the distance from the point K_n, passed by the wheel, to B. If the decrease of (11) by the quantity $W_n \frac{b_1}{p}$ changes the sign of (11), then S_1 reaches a maximum, and in general its greatest maximum with W_n at K_n. If, however, (11) does not become negative by this decrease, the train must be still further moved toward the left until some wheel passes a joint of component I which changes the sign of (11), or until some wheel passing C makes (11) negative. When joint C is in the left half of the span the discussion differs from that just given for the right half only in this, that the possibility of a maximum when some wheel passes C is shut out, as was shown previously.

Secondly, in case (11) is negative when W_1 stands just at the right

Digitized by Google

of C, then (11) must have become negative before the train reached this position, which could have occurred only as some wheel W_n passed same joint K_n of component I very shortly before W_1 reached C. Move the train backward until it first occurs that a wheel passes a joint of component I. Call the joint K_n and the wheel weight W_n, and see whether the decrement $W_n \frac{b_1}{p}$ due to the passage of W_n across K_n from right to left has been the cause of the negative value of (11). If the wheel W_n is one near the head of the train, it is in general the one which has rendered (11) negative; but if the wheel is one passing a joint near B, it is not so likely to be the one which has rendered (11) negative. By continuing the examination, the wheel will soon be discovered which has rendered (11) negative. It is taken for granted in this discussion that the head of the train will not arrive at the joint of component II next left of H before the greatest value of S_1 has been reached. If in any exceptional case W_1 should arrive at the joint next to the left of H before S_1 reaches a maximum, it might then produce a maximum of S_1 by passing that joint, for it would increase P_2 and decrease P_1 at that joint and so tend to make (11) negative.

Furthermore, in considering the position of W_1 with respect to C, it will be noticed that in case the joint C is one situated near B, it is impossible for W_1 to pass C without making (11) negative, though it may do so in case C is a joint situated near A. Besides this, it appears that the value of the quantity $(P_2 - P_1)$ at a joint K near the head of the train, has a more decisive effect in controlling the sign of δS_1 when C is near B than when C is near A, and this alteration goes on from joint to joint as we proceed from B to A. The total effect of this is to allow the head of the train to stand a little further to the left with respect to C, the further C is toward the left. This agrees with the result already found to hold for the simple girder, and for the single intersection truss, in both of which the train advances with respect to the cross section, at which the maximum shear occurs as the cross section advances from B toward A. The advance here spoken of takes place in each case by jumps from wheel to wheel; but in the double intersection truss, as just shown, the law is more complicated than in the other two cases, as the maximum shear depends upon other wheels besides the first one or two at the head of the train. This prevents our finding the segment in which the greatest shear is dominated by any particular wheel, though it might be possible to determine this approximately. Such a determination seems, however, to be nearly useless in view of what has been just stated as to the continued advance of the train with respect to C. For when the train position has been found which makes the shear a maximum at joint 1, then the maximum shear at joint 2 is due either to a train position which is the same relatively to joint 2, or to a position slightly in advance of this; and so on for joint 3, etc. It is to be specially

Digitized by Google

noticed that the investigation which has been made of the train position for maximum shear in the web members of any panel has been entirely independent of any assumption as to pattern of the truss, except that it consists of two simple trusses superposed in such a manner that the equidistant joints belong alternately to each of the components. This has important consequences, viz.: the investigation not only applies to the form of truss shown in Fig. 7, but to any modifications of it, obtained by changing the inclination of any or all of the web members. For example, it applies to the case where the end posts are inclined, according to the usual design as shown in the dotted lines, Fig. 7. Again, the ordinary double intersection Whipple Truss with inclined end posts and vertical intermediate posts, is also included in our investigation, provided we may assume the vertical tie at the first panel point to belong exclusively to one component, as we are entitled to do without appreciable error, and we also make the like assumption respecting the vertical tie at the last panel point.

Likewise the Post Truss, or any other form of single intersection truss is also included in equation (11). The only point to be considered is whether the maximum shear, as thus determined, is to be sustained by principal web members or by counters; the latter is the case when the shear in one half of the span is of the same sign as that in the end post of the other half span.

11. The train position which produces a maximum stress in the web members of a multiple intersection truss may also be discovered by equation (11).

This is evident when we notice that (11) was derived by taking into account only those pin loads designated as R_1 which are supported by component I, and it is of no consequence how many other components there may be in the complete truss, as (11) is independent of all of them.

Hence in applying equation (11) to discover the position of the train for a maximum value of S_1 in a multiple intersection truss it will be necessary to take the value of $(P_2 - P_1)$ at each panel point only of the component whose web member we are considering, and effect the summations expressed in equation (11) for that component alone. In the multiple truss there will be several panel points of other components intermediate between successive panel points, as H and K, of the component under consideration; but in taking the summations in equation (11) it will make no difference which of the intermediate points be regarded as C.

Suppose, for example, that the intermediate panel point nearest H is denoted by C_1, and that nearest K by C_2, then the first summation of equation (11) may conveniently be taken from A to C_1 and the second from C_2 to B.

If the panel points of the truss be marked upon the edge of a strip of

Digitized by Google

paper the computation of equation (11) can be made as before, but with this difference, that while in a double intersection truss all the wheel weights upon the truss enter either one or other of the summations, in any other multiple intersection truss only those wheel weights are included in the summations which stand in the panels adjacent to the panel points of the component under consideration. The wheel weights upon other panels than these have no influence upon the value of S_1.

The circumstances under which the last member of (11) may change sign from positive to negative, and so cause a maximum value of S_1, are these: Some wheel, as W_n, may pass one of the panel points of component I between C_2 and B, and so decrease (11) by $W_n \frac{b_1}{p}$, or, some wheel W_n may enter upon or leave some panel included in the summation between A and C_1, and so decrease (11) by $W_n \cdot \frac{a_1}{p}$.

It will be noticed when any wheel enters upon or leaves any panel included in the summation between C_2 and B that it tends to make (11) positive; and when any wheel passes a panel point of I between A and C_1 it has a like tendency.

The greatest value of S_1 will be found when the head of the train is at or near C_1, and its precise position can be discovered by a method like that previously explained as applicable to the double intersection truss. It will only be necessary to designate each component as it successively comes under consideration as component I.

Remarks like those made at the end of the previous article are also true for the multiple intersection truss, viz.: the pattern of the truss is of no account, provided we may assume with sufficient correctness that the pin loads of one component are not transmitted to the piers through the web members of the other components.

12. To discover the train position which produces a maximum shearing stress in a web member of a double intersection truss with intermediate supports *O I*, *O J*, etc., Fig. 9.

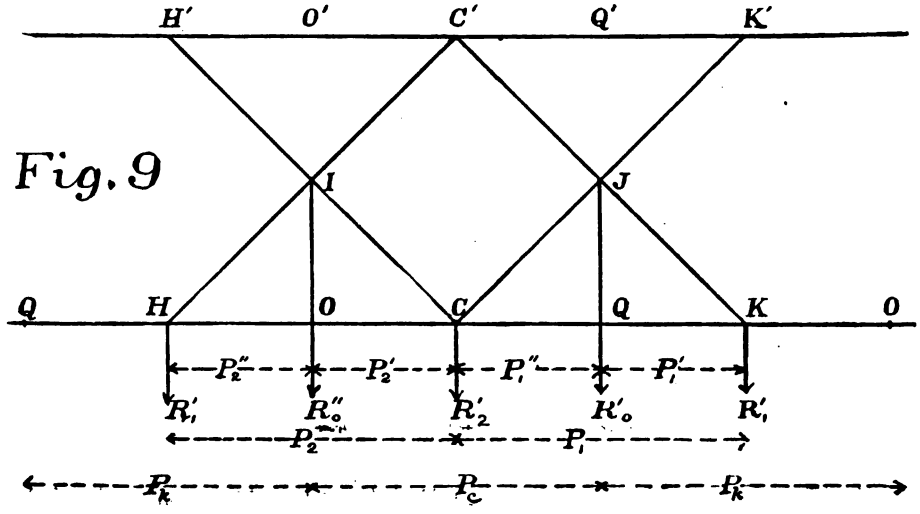
The double intersection truss with intermediate supports, as shown in Fig. 9, is a design so commonly employed and so economical of material, that we proceed to investigate the train positions for maximum stresses in the web members by a method similar to that just employed for the double intersection truss.

As this is a modification of the double intersection truss previously treated, let the notation previously employed be retained with certain slight modifications and additions.

Designate the loads at the pin points *H*, *K*, etc., of component I by R_1', those at *C*, etc., of II by R_2' and those at the intermediate pin points next at the left of each R_1'' (as at *Q*, etc.) by R_0', but those next left of each R_2' (as at *O*, etc.) by R_0''. Furthermore, let the panel load P_2 upon *H C* consist of two parts, P_2'' upon *H O* and P_2' upon *O C*;

Digitized by Google

Fig. 9



Digitized by Google

and similarly for P_1 upon $C\,K$, so that $P_2 = P_2'' + P_2'$ and $P_1 = P_1'' + P_1'$. Moreover, let the load upon $O\,Q$ be $P_c = P_2' + P_1''$, then is P_c the load in the two half panels about any point C of II, and similarly let $P_k = P_1' + P_2''$ be the load in the two half panels about any panel point H or K of I.

Then, by our previous notation for distances, taken as positive toward the left, and by taking $H\,C = p = C\,K$ and $O\,C = \frac{1}{2}\,p = C\,O$, etc., we have the equations,

$$\begin{aligned}
\tfrac{1}{2}\,p\,R_0'' &= P_2'\,c - P_2''\,h \\
\tfrac{1}{2}\,p\,R_0' &= P_1'\,k - P_1''\,c \\
\tfrac{1}{2}\,p\,R_1' &= P_2''\,o - P_1\,q \\
\tfrac{1}{2}\,p\,R_2' &= P_1''\,q - P_2'\,o
\end{aligned}$$

1° The shear in the web members of component I at any point between O and Q may evidently be written in the form

$$S_1 = V_1 - \sum_A^C [R_1' + \tfrac{1}{2}\,(R_0' + R_0'')],$$

for R_1' is entirely supported by I, and by reason of the symmetry of the web members at I and J, each intermediate pin load R_0' and R_0'' is supported half by component I and half by II.

By taking moments about B we find the load upon component I causes a reaction at A to the amount

$$V_1 = \frac{1}{l}\sum_A^B [(R_1'\,b_1 + \tfrac{1}{2}\,R_0'\,(b_1 + \tfrac{1}{2}\,p) + \tfrac{1}{2}\,R_0''\,(b_1 - \tfrac{1}{2}\,p)].$$

Substitute this value of V_1 in the previous equation and we have

$$2\,S_1 = \frac{1}{l}\sum_A^B [(2\,R_1' + R_0' + R_0'')\,b_1 + \tfrac{1}{2}\,(R_0' - R_0'')\,p] - \sum_A^C [2\,R_1' + R_0' + R_0''].$$

In this equation introduce the values of the various pin-loads R_1', R_0', R_0'', previously given, and then let the train move a small distance $\delta\,x$ toward the left.

We then finally obtain the equation :

$$p\,\frac{\delta\,S_1}{\delta x} = \frac{1}{l}\sum_A^B [(P_2 - P_1)\,b_1 + \tfrac{1}{2}\,(P_k - P_c)\,p] - \sum_A^C (P_2 - P_1),$$

or,

$$p\,\frac{\delta\,S_1}{\delta\,x} = \sum_A^C (P_2 - P_1)\,\frac{a_1}{l} - \sum_C^B (P_2 - P_1)\,\frac{b_1}{l} + \frac{p}{2\,l}\,(P_k - P_c) \ldots (12)$$

Digitized by Google

By comparing this equation with equation (11), it appears that it differs from (11) only by the addition of a final term which will, under all ordinary circumstances, be small. The presence of this term in (12) makes it possible, however, that the maximum value of S_1 may occur as some wheel passes any one of the points Q, but not as it passes any of the points O. S_1 may also be a maximum in (12) under all the same circumstances as S_1 in equation (11), which need not be here repeated.

2°. To find the value of S_1 between Q and K, *i. e.*, in JK, it is necessary to subtract the quantity $\frac{1}{2} R_0'$ at Q from the value of S_1 already found between O and Q.

Hence we must subtract from (12) the quantity

$$\frac{1}{2} p \frac{\delta R_0'}{\delta x} = \sum_{C}^{K} (P_1' - P_1'')$$

This term of correction, whose value may be large, when appended to equation (12) makes it evident that the maximum value of S_1 in JK may occur just as some wheel passes K.

3°. Similarly to find the value of S_1 between H and O, *i. e.*, in HI, it is evidently necessary to add the quantity $\frac{1}{2} R_0''$ at O to the value of S_1 already found between O and Q. Hence we must add to (12) the correction

$$\frac{1}{2} p \frac{\delta R_0''}{\delta x} = \sum_{H}^{C} (P_2' - P_2'')$$

which makes it evident that the maximum value of S_1 in HI may occur just as some wheel passes O.

13. To discover the train position which produces the greatest stress in any member of the loaded chord of a double intersection truss with parallel chords, as in the bar CK, Fig. 7.

Let the notation be like that employed in connection with Fig. 7, viz.:

R_1 = the load at any pin point of I,
R_2 = the load at any pin point of II,
l = length of span; h = depth of truss,
c' = the distance of any R_1 from C',
k' = the distance of any R_2 from K',

these distances being both taken as positive toward the left,

$$c_a' = C'A, \quad k_a' = c_a' + p = K'A.$$

T_1 = tension in chord member of I opposite the joint C'.
T_2 = tension in chord member of II opposite the joint K'.
$T = T_1 + T_2$ = total tension in CK.

Take moments about C' of all the forces acting on component I at the left of C', and we have

$$h T_1 = V_1 c_a' - \sum_{A}^{C'} R_1 c'$$

Digitized by Google

Similarly take moments about K' of all forces acting on component II, and we have

$$h\,T_2 = V_2\,k_a' - \sum_A^{K'} R_2\,k'$$

in which equations the reactions at A of components I and II are respectively

$$V_1 = \frac{1}{l}\sum_A^B R_1\,b_1,\quad V_2 = \frac{1}{l}\sum_A^B R_2\,b_2,$$

b_1 and b_2 being distances of R_1 and R_2 respectively from B. Add the two equations of moments just given, replace $T_1 + T_2$, V_1 and V_2 by their values already given, and finally substitute for R_1 and R_2 their values from the equations

$$p\,R_1 = P_2\,c_2 - P_1\,c_2,\quad p\,R_2 = P_1\,k_1 - P_2\,k_2.$$

The value of R_1 has already been obtained in connection with Fig. 8. The value of R_2 is derived in a similar manner, as will be evident when we notice in Fig. 8 that the panel loads about K in component I follow each other from left to right in the order $P_1\,P_2$, so that the value of R_1 is obtained from consideration of three consecutive panel points in the order $C_1\,K\,C_2$, while the panel loads about any panel point, C in component II follow in the reverse order $P_2\,P_1$; hence the value of R_2 is obtained from the consideration of three consecutive panel points in the order $H\,C\,K$; or, taking a more systematic notation, we may use instead the letters $K_2\,C\,K_1$. The equation finally obtained may be written in the form

$$p\,h\,T = \frac{c_a'}{l}\sum_A^B (P_2\,c_2 - P_1\,c_1)\,b_1 - \frac{k_a'}{l}\sum_A^B (P_2\,k_2 - P_1\,k_1)\,b_2$$
$$- \sum_A^{C'} (P_2\,c_2 - P_1\,c_1)\,c' + \sum_A^{K'} (P_2\,k_2 - P_1\,k_1)\,k'.$$

Now let the train move a small distance $\delta\,x$ toward the left, then will each of the distances c_1, c_2, k_2, k_2 be increased by $\delta\,x$.

Let T be changed by this movement to $T + \delta\,T$; we then obtain by subtraction and division,

$$p\,h\,\frac{\delta\,T}{\delta\,x} = \frac{c_a'}{l}\sum_A^B (P_2 - P_1)\,b_1 - \frac{k_a'}{l}\sum_A^B (P_2 - P_1)\,b_2 - \sum_A^{C'} (P_2 - P_1)c'$$
$$+ \sum_A^{K'} (P_2 - P_1)\,k'.$$

In order to trace changes in the sign of this equation more readily, let us make certain transformations.

Digitized by Google

Since b_1 refers to R_1 and not to R_2, it is evident that the b_1 of any P_1 extends from B to the right hand extremity of the panel in which this P_1 stands; but, since b_2 refers to R_2 only, the b_2 of any P_1 extends from B to the left extremity of the panel in which P_1 is, and *vice versa*, for the b_1 and b_2 of any P_2.

Hence	for	any	P_1 we have	$b_2 - b_1 = p,$
but	"	"	P_2 " "	$b_1 - b_2 = p.$
Similarly	"	"	P_1 " "	$k' - c' = 2p,$
but	"	"	P_2 " "	$k' - c' = 0.$

Use these equations to eliminate b_2 and c', and put $c_a' = k_a' - p$.

The result may be written thus:

$$p\,h\frac{\delta T}{\delta x} = \frac{k_a' - p}{l}\sum_A^B (P_2 - P_1)\,b_1 - \frac{k_a'}{l}\sum_A^B [P_2\,(b_1 - p) - P_1\,(b_1 + p)] - \sum_A^{C'} [P_2\,k' - P_1\,(k' - 2p)] + \sum_A^{C'} (P_2 - P_1)\,k' + \sum_{C'}^{K'} (P_2 - P_1)\,k',$$

in which the last two summations take the place of the last one in the previous equation by help of the identity, $A\,K' = A\,C' + C'\,K'$. Cancel and divide by p.

$$\therefore\ h\,\frac{\delta T}{\delta x} = \frac{k_a'}{l}\sum_A^B (P_2 + P_1) - \sum_A^B (P_2 - P_1)\,\frac{b_1}{l} - 2\sum_A^{C'} P_1 + \sum_{C'}^{K'} (P_2 - P_1)\,\frac{k'}{p}.$$

But we have identically

$$\sum_A^{C'} (P_2 - P_1)\,\frac{(b_1 - a_1)}{l} - \sum_A^{C'} (P_2 + P_1) = -2\sum_A^{C'} P_1,$$

by help of which our equation is reduced to the form

$$h\,\frac{\delta T}{\delta x} = \frac{k_a'}{l}\sum_A^B (P_2 + P_1) - \sum_A^{C'} (P_2 + P_1) + \sum_{C'}^{K'} (P_2 - P_1)\,\frac{k'}{p} - \sum_A^{C'} (P_2 - P_1)\,\frac{a_1}{l} - \sum_{C'}^B (P_2 - P_1)\,\frac{b_1}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

This is a general equation for a member of the loaded chord, and applies to a double intersection truss of any pattern. Let us first apply it to the truss shown in Fig. 7. In this case we may omit all primes, for $k = k'$, etc., and C and C' are on the same vertical, etc.,

Digitized by Google

Since $P_2 = 0$ between C' and K', we also have in this case,

$$\sum_{C}^{K'} (P_2 - P_1) \frac{k'}{p} = -\sum_{C}^{K} P_1$$

The result after reduction may be written in the form—

$$h \frac{\delta T}{\delta x} = \frac{k_a}{l} \sum_{A}^{B} (P_2 + P_1) - \sum_{A}^{K} (P_2 + P_1) - \sum_{A}^{C} (P_2 - P_1) \frac{a_1}{l}$$

$$- \sum_{C}^{B} (P_2 - P_1) \frac{b_1}{l},$$

or more briefly—

$$h \frac{\delta T}{\delta x} = k_a \left[\frac{L}{l} - \frac{K_a + C_1}{k_a} \right] \ldots\ldots\ldots\ldots (13_1)$$

in which equation as before—

L = the total load on the span $A\,B$.
K_a = " " " segment $K\,A$.

Also,

$$C_1 = \sum_{A}^{C} (P_2 - P_1) \frac{a_1}{l} + \sum_{C}^{B} (P_2 - P_1) \frac{b_1}{l} = -\frac{2}{l} (A_1 - B_1),$$

in which A_1 and B_1, have the values ascribed to them in Article 10, which are in general numerically small and separately vanish when the load is uniformly distributed. We may then regard C_1 as a small correction to be applied to K_a, and if it be disregarded for the moment, equation (13_1) would evidently give a train position identical with that before found to produce a maximum bending moment at K in a simple girder $A\,B$ in case $A\,K = A\,K' = k_a$. Now, in order to discover under what circumstances the last member of (13_1) may change sign from positive to negative, and so make T a maximum we must learn under what circumstances the last fraction in (13_1) receives a positive increment.

In case the joint K is in the right half of the span, let any wheel as W_u move from right to left of K, then is the quantity K_a increased by the wheel weight W_u, and the quantity C_1 is at the same time decreased by the amount $2\,W_u \frac{b_1}{l}$ (in which $b_1 = B\,K$), by reason of the passage of W_u from P_2 to P_1 in passing K. Hence the last numerator of (13_1) is increased on the whole by the amount—

$$W_u \frac{(l - 2\,b_1)}{l}.$$

Digitized by Google

This quantity is positive in the right half of the span, for we then have $l > 2\,b_1$, hence T may reach a maximum in the right half of the span just as some wheel W_u passes K, as it would not were K in the left half of the span.

In case the joint C is in the left half of the span, let any wheel as W_u pass from right to left of C, then will the left P_1 which is included in B_1, be decreased by W_u, and the right P_2 in A_1 will be increased by W_u. Hence the last numerator in equation (13_1) is increased on the whole by the quantity $W_u \frac{b_1 + a_1}{l}$ in which a_1 is negative, but $b_1 + a_1 > 0$, since C was taken in the left half of the span.

It is then evident that T may reach a maximum value in the left half of the span just as some wheel passes C, as it would not in the right half of the span.

Again, without regard to the right or left half of the span let any wheel as W_u pass some joint of component II at the right of K. Then W_u passes over from the P_1 at the right of this joint to the P_2 at its left, and at the same instant its b_1 is increased by $2p$. Hence such a movement suddenly increases C_1 by the quantity $2\,W_u \frac{p}{l}$, and this regardless of which joint of II the passage occurs at, provided only it is at the right of K. From this it is evident that T may reach a maximum just as a wheel passes any joint of II at the right of K, as it does not when it passes any joint of I at the right of K, for this latter change tends only to decrease B_1.

Once more, without regard to the position of the joint C, let some wheel as W_u, move from right to left of any joint of I which is situated at the left of C, then W_u passes over from P_2 at that joint to P_1. Hence such a movement suddenly increases the last numerator by the numerical amount $2\,W_u \frac{a_1}{l}$, an amount which is larger the nearer the joint is to C. From this we see that T may also reach a maximum value when a wheel passes any joint of I at the left of C, but is more likely to do so, other things being equal, the nearer the joint is to C.

Since the quantity $A_1 - B_1$ appears in equations (11) and (13_1) with opposite signs, the conclusion drawn from (11) to the effect that the train progresses with respect to the panel where the shear is a maximum, must be drawn from equation (13_1) in the reverse way as to the chord stress T, to the effect that the train retrogrades on the whole with respect to the panel $C\,K$ in which the chord stress is a maximum as that panel is assumed further toward the left.

In a general way it may be said that the greater concentration of load is forward, *i. e.*, to the left of the panel under consideration in the right half of the span and behind it, *i. e.*, to the right of it, in the left half, or in other words, it is nearer the middle of the span than the panel is, at the

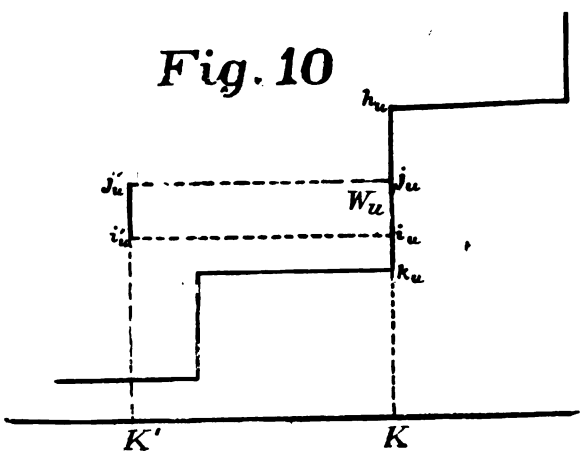
Digitized by Google

instant when the greatest stress occurs in the chord member of the panel.

We have then the following method of procedure for finding the train position which produces a maximum stress in $C\,K$.

1°. In case K is in the right half of the span, find by help of the weight line, as previously explained, the wheel W_u which, standing at K, would produce the greatest bending moment at K in a simple girder $A\,B$. The train position thus found is approximately that which produces a maximum stress in $C\,K$. Take W_u as standing just at the right of K and compute the value of C_1 for this train position. It will be shown a little later how this computation may be most expeditiously made by help of a graphical construction. The value thus obtained for C_1 is a positive or negative correction of K_a, and accordingly is to be laid off on the diagram of the weight line upward or downward from the lower extremity k_u of W_u. Let $k_u\,i_u$ be this correction in Fig. 10.

Fig. 10



Now let W_u pass K from right to left, then will the last numerator of (13_1) be increased by the quantity

$$W_u\,\frac{l-2\,b_1}{l}$$

due to the passage of W_u across the point K. Let $i_u\,j_u$ represent this quantity in Fig. 10. Then may $i_u\,j_u$ be regarded as a fictitious weight such that, when it is transferred to the vertical at K' and used in the same manner as we have previously used actual weights for discovering maxima, it will enable us to discover the train position for a maximum value of T. In case K and K' are in the same vertical no transference is needed. Let the transferred position be $i_u'\,j_u'$, then it constitutes the fictitious weight in its true position on the diagram of the weight line, and when it is intersected between its extremities i_u' and j_u' by the slope

Digitized by Google

line $L \div l$ such intersection shows the train position to be one producing a maximum chord stress T in $C\,K$, but if not, not. This statement is evident since such intersection shows that (13_1) changes sign as W_u passes K. In case the slope line does not intersect the fictitious weight in the assumed position of the train, the train must be moved to the right or left until such intersection occurs just as some wheel passes some joint of II at the right of K, or some joint of I at the left of C.

It is more probable that the train must in this case be moved toward the left than toward the right, for reasons already given. On moving the train, whenever a wheel arrives at either of the joints just mentioned as possibly making T a maximum, a recomputation of (13_1) must be made. This will, in general, give the fictitious weight $i_u'\,j_u'$ a magnitude and position different from that before obtained.

Whenever these are such that it is cut between its extremities by the slope line $L \div l$, then the train has arrived at a position where it makes T a maximum. If there is more than one such position, the computed numerical values of T may be directly compared to discover which is greatest, as their general comparison algebraically appears too complicated to be of practical use.

2°. In case C is taken in the left half of the span, the method of procedure is the same as that just developed, except that it may be well to take as the initial position that which produces the maximum bending moment at C in a simple girder of span $A\,B$.

As shown before, the numerical value of $i_u'\,j_u'$ due to the passage of the weight W_u from right to left of C is

$$W_u\,\frac{a_1+b_1}{l}$$

If this fictitious weight is not intersected by the slope line $L \div l$, and so a maximum value of T does not occur in $C\,K$ as W_u passes C, then the train must be moved to the right or left and the proper train position found as before, but the train will more probably require to be moved to the right than to the left. But any uncertainty in this particular is mostly obviated in actual computation by finding the required train position for the panels successively from end to end of the truss.

For if the train position be found which produces a maximum chord stress in the left panel nearest A, say in $C_1\,K_1$, then, as previously stated, the train position which gives a maximum in the next panel $C_2\,K_2$ at the right of $C_1\,K_1$ is such that the train may stand in general a little further to the left with respect to $C_2\,K_2$ than it did with respect to $C_1\,K_1$ when it produced a maximum in $C_1\,K_1$, though this is of course not so far to the left with respect to $A\,B$. Hence, if W_u at C_1 makes T a maximum in $C_1\,K_1$, and W_u at C_2 does not make T a maximum in $C_2\,K_2$ then W_u must in general be moved toward the left to reach the train position which makes T a maximum in $C_2\,K_2$, and so on for the other panels of this half of the truss.

Digitized by Google

The graphical computation of C_1 may be conveniently arranged, as shown in Fig. 11. Lay off on a piece of cross-section paper the span $A\,B$ and the panels $C\,K$, etc., of the truss to the same scale as the diagram of the wheel distances.

Then for any position of the truss at which it is desired to compute C_1 read off the value of $P_2 - P_1$ at each joint of I by the graphical method already given.

Read on the same scale or transfer directly, with compasses, the weight

$$P_2 - P_1 = A\,y_a$$

taken as positive upwards. Draw the thread or line $B\,y_a$; then is

$$(P_2 - P_1)\,\frac{b_1}{l} = K\,y_k$$

which can be read off to scale. In similar manner find at H

$$(P_2 - P_1)\frac{a_1}{l} = H\,y_h$$

which is to be taken downwards and negative when $P_2 - P_1 > 0$, because a_1 is essentially negative, and *vice versa*.

In this way values of all the quantities in the two summations which form C_1 in equation (13_1) may be found. After C_1 has been evaluated it can be used as proposed in the method already explained.

It is evident that in obtaining equation (13), b_1, k' and k_a' might have been eliminated instead of $b_2\,c'$ and c_a'. We should thus have been led by a similar series of transformations to the following general equation equivalent to (13):

$$h\,\frac{\delta\,T}{\delta\,x} = \frac{c_a'}{l}\sum_{A}^{B}(P_2 + P_1) - \sum_{A}^{C'}(P_2 + P_1) + \sum_{C'}^{K'}(P_2 - P_1)\,\frac{k'}{p}$$

$$- \sum_{A}^{C'}(P_2 - P_1)\,\frac{a_2}{l} - \sum_{C'}^{B}(P_2 - P_1)\,\frac{b_2}{l} \quad \ldots\ldots\ldots (14)$$

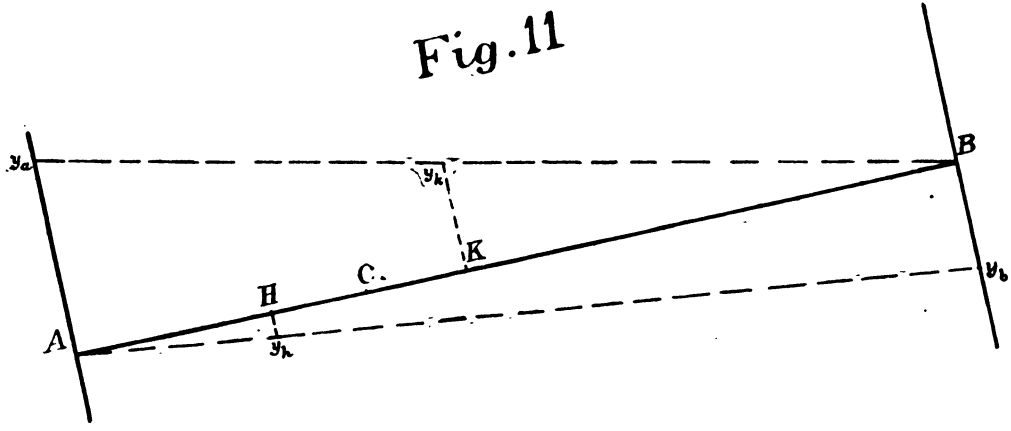
which may be more convenient than equation (13) for graphical construction in some cases.

Equation (14) may be adapted to the truss in Fig. 7 by noticing that $\frac{k'}{p} = 1$ between C' and K', and making use also of the identity

$$\sum_{C'}^{K'}(P_2 - P_1) = \sum_{C}^{K}(P_2 - P_1)\,\frac{b_2 - a_2}{l}$$

Digitized by Google

Fig. 11



Digitized by Google

we obtain

$$h\frac{\delta T}{\delta x} = \frac{c_a}{l}\sum_{A}^{B}(P_2 + P_1) - \sum_{A}^{C}(P_2 + P_1)$$

$$- \sum_{A}^{K}(P_2 - P_2)\frac{a_2}{l} - \sum_{K}^{B}(P_2 - P_1)\frac{b_2}{l},$$

or

$$h\frac{\delta T}{\delta x} = c_a\left[\frac{L}{l} - \frac{C_a + K_2}{c_a}\right] \ldots\ldots\ldots\ldots\ldots (14_1)$$

which equation would serve to discover the set of conditions already found by means of (13_1).

14. To discover the train position which produces the greatest stress in any member of the unloaded chord of a double intersection truss, as in the bar $C'K'$ of Fig. 7:

Take the moment about K of all the forces acting on component I, and that about C of all the forces acting on II and we have

$$h\,T''_1 = V_1\,k_a - \sum_{A}^{C} R_1\,k$$

$$h\,T'_2 = V_2\,c_a - \sum_{A}^{H} R_2\,c$$

Now $T'' = T''_1 + T'_2$ and $c_a = k_a - p$, in which equations primes are used to refer to the unloaded chord; also k is the distance of any R_1 from K and c that of any R_2 from C.

Adding the equations of moments and substituting the values of the reactions and pin loads as was done in the previous case we have

$$p\,h\,T'' = \frac{k_a}{l}\sum_{A}^{B}(P_2\,c_2 - P_1\,c_1)\,b_1 - \frac{c_a}{l}\sum_{A}^{B}(P_2\,k_2 - P_1\,k_1)\,b_2$$

$$- \sum_{A}^{C}(P_2\,c_2 - P_1\,c_1)\,k + \sum_{A}^{H}(P_2\,k_2 - P_1\,k_1)\,c,$$

Move the train a small distance δx toward the left,

Then	for each	P_1,	$k - c$	$= 0$
but	" "	P_2,	$k - c$	$= 2$
while	" "	P_1,	$b_2 - b_1$	$= p$
but	" "	P_2,	$b_1 - b_2$	$= p$

By eliminating b_1 and $c_a = k_a - p$ we find after reductions like those used in obtaining equation (13)

Digitized by Google

$$h\frac{\delta T'}{\delta x} = \frac{k_a}{l}\sum_{A}^{B}(P_1 + P_2) - \sum_{A}^{H}(P_1 + P_2) + \sum_{H}^{C}(P_1 - P_2)\frac{k}{p}$$

$$- \sum_{A}^{H}(P_1 - P_2)\frac{a_2}{l} - \sum_{H}^{B}(P_1 - P_2)\frac{b_2}{l} \quad \ldots\ldots\ldots (15)$$

If, however, b_2 and k_a be eliminated the result obtained is,

$$h\frac{\delta T'}{\delta x} = \frac{c_a}{l}\sum_{A}^{B}(P_1 + P_2) - \sum_{A}^{H}(P_1 + P_2) + \sum_{H}^{C}(P_1 - P_2)\frac{k}{p}$$

$$- \sum_{A}^{H}(P_1 - P_2)\frac{a_1}{l} - \sum_{H}^{B}(P_1 - P_2)\frac{b_1}{l} \quad \ldots\ldots\ldots (16)$$

Equations (15) and (16) are general and apply to any double intersection truss.

Equation (15) may be adapted to the truss in Fig. 7, by noticing that in the third term $\frac{k}{p} = 2$ and $P_1 = 0$ between H and C, also $P_2 = 0$ between C and K. Hence we may write the identities

$$2\sum_{H}^{C}(P_1 - P_2) = \sum_{H}^{C}(P_1 - P_2)\frac{b_2 - a_2}{l} - \sum_{H}^{C}(P_1 + P_2),$$

$$0 = \sum_{C}^{K}(P_1 - P_2)\frac{(b_2 - a_2)}{l} - \sum_{C}^{K}(P_1 + P_2).$$

Hence equation (15) becomes

$$h\frac{\delta T'}{\delta x} = \frac{k_a}{l}\sum_{A}^{B}(P_1 + P_2) - \sum_{A}^{K}(P_1 + P_2)$$

$$- \sum_{A}^{K}(P_1 - P_2)\frac{a_2}{l} - \sum_{K}^{B}(P_1 - P_2)\frac{b_2}{l},$$

or,

$$h\frac{\delta T'}{\delta x} = k_a\left[\frac{L}{l} - \frac{K_a - K_2}{k_a}\right] \ldots\ldots\ldots\ldots\ldots (15_1)$$

in which the notation is obvious.

Equation (16) may be similarly transformed to

$$h\frac{\delta T'}{\delta x} = \frac{c_a}{l}\sum_{A}^{B}(P_1 + P_2) - \sum_{A}^{C}(P_1 + P_2)$$

$$- \sum_{A}^{C}(P_1 - P_2)\frac{a_1}{l} - \sum_{C}^{B}(P_1 - P_2)\frac{b_1}{l},$$

or,

$$h\frac{\delta T'}{\delta x} = c_a\left[\frac{L}{l} - \frac{C_a - C_1}{c_a}\right] \ldots\ldots\ldots\ldots\ldots (16_1)$$

Digitized by Google

These equations are to be interpreted in a manner similar to (13_1) and determine position of the train which produces a maximum in $C'K'$ of Fig. 7, in the manner already discussed in connection with that equation.

15. To discover the train position which produces a maximum stress in any chord member of the double intersection truss with intermediate supports, Fig. 9:

The expression for the total tension T in any member CK, Fig. 9, differs from that already obtained for the stress in the same member CK, Fig. 7, only in terms depending upon the magnitude and position of the wheel-weights between C and K. The truth of this statement may be made to appear as follows: the total reaction V at the pier A depends solely upon the magnitude and position of the wheel-weights between A and B, and not upon the arrangement of the members of the truss. Now take the moments about Q' situated in the vertical at J of all the external forces applied to the truss at the left of Q'. So far as V and all the wheel-weights at the left of C are concerned we are at liberty in computing their moments about Q' to take either the wheel-weights themselves or the pin loads; but in considering the wheels between C and K it is necessary to take into account how much of this panel load is supported at C.

It is assumed that this has been done correctly and has been included in the value of $h\,T$ obtained in the case of Fig. 7, and that we should find the same value of $h\,T$ in Fig. 7 if it were computed by taking moments about any point Q' between C' and K' of all the forces applied at the left of Q', provided we should also correctly take account of the stresses in the web members of the panel. It was the difficulty of attending properly to this last matter that has led to the method of separately computing T' and T'', and then taking their sum to obtain T.

The value of $h\,T$ is, then, equal to the algebraic sum of the moments of all the forces at the left of Q' minus the moment about Q' of the stresses in the members of the web cut by the vertical at Q'. As already stated, the moment of the applied forces differs in the two cases only in so far as the panel load CK is concerned, and the moment of the stresses in the web members differs not at all in the two Figs. 7 and 9. This last assertion may be seen to be true from the following considerations. Whatever changes are produced in the stresses of the web members meeting at J, when Q is supported from J, it is evident from their symmetrical disposition about the vertical at J, that the part of the load applied at J, which is transmitted to the pier at A causes as great a tension in $C'J$ as compression in CJ. Hence their moment of resistance about Q' vanishes. Now, that part of the value of $h\,T$ in Fig. 7, which depends upon the panel load of CK is the part of the pin load at C due to this panel, which is $\frac{P_1\,k}{p}$, multiplied by its lever

Digitized by Google

arm from Q, which $Q\,C = \frac{1}{2}\,p$. Hence the part of $h\,T$, in Fig. 7, due to the panel $C\,K$, is $\frac{1}{2}\,P_1\,k$.

Again, that part of $h\,T$, in Fig. 9, which depends upon the panel load of $C\,K$ is that part of the pin load at C due to this panel, which is $\frac{P_1'\,q}{\frac{1}{2}\,p}$, multiplied by its arm, $\frac{1}{2}\,p$, or, in all, $P_1'\,q$. The whole effect will then be to replace the last term in the value $p\,h\,T$ in Art. 13, viz.:

$$\sum_{A}^{K'} (P_2\,k_2 - P_1\,k_1)\,k'$$

by the following expression:

$$\sum_{A}^{C} (P_2\,k_2 - P_1\,k_1)\,k' - \sum_{C}^{Q} P_1'\,q\,k'$$

so that we must then, in equation (13), replace

$$K_a = \sum_{A}^{K} (P_2 + P_1) = \sum_{A}^{K} W$$

by the following:

$$Q_a = \sum_{A}^{Q} W$$

The introduction of Q_a instead of of K_a will simply have the effect of making the point Q play the rôle ascribed in the discussion of equation (13_1) to the point K.

It will be seen from these expressions that the introduction of intermediate pin points has the effect to increase slightly the tension of the loaded chord for a given train position; but at the same time it materially decreases the bending moments of the stringers, since they are decreased in length to $\frac{1}{2}\,p$. The stresses in the web members are also slightly increased. There is also an increase of material due to the introduction of the pin points at Q and J and the pieces of the type $Q\,J$.

The stress T' in any member $C'\,K'$ of the unloaded chord is unchanged by the introduction of intermediate joints in the loaded chord.

16. To discover the position of the train which produces the greatest stress in any given bar of the loaded chord of a double intersection truss with vertical web members, as in the Whipple truss.

Since the stress is identical in the members of the upper and lower chords respectively, which lie between any pair of consecutive diagonals, it will in general be possible to determine the train position for a maximum stress in any bar of either chord from either of the equations (13), (14), (15) or (16).

In trying to determine the train position for a Whipple Truss, which possesses a system of counters as well as principal web members, it might not at first appear which system should be considered as acting.

Digitized by Google

But in case the stress in a bar of the loaded chord is at a maximum it is sufficiently evident that the principal web members between that bar and the nearest pier are in action, a statement which will be seen to be in accordance with that made in connection with Fig. 7, where it was shown that to produce a maximum stress in a bar the greatest concentration of load lies between it and the center of the span, which is true in general for any truss with end supports.

1°. In order to find the train position for a maximum stress in $C\,K$ when C is in the left half of the truss, let the inclinations of the web members shown in Fig. 7 be changed until $H\,C'$ and $C\,K'$ become vertical, etc. The reader will readily supply the appropriate figure. Now apply the general equation (13) to this case, in which we first replace the limit C' by K and $k_a' = c_a$ and notice that the third term vanishes since $k' = 0$ between C' and K', *i. e.*, between H and C; also add the identity

$$0 = \sum_{H}^{C} (P_2 - P_1)\,\frac{b_1 - a_1}{l} - \sum_{H}^{C} (P_2 + P_1),$$

which evidently is true because $P_1 = 0$ between H and C.

After reductions such as we have made before we have

$$h\,\frac{\delta\,T}{\delta\,x} = \frac{c_a}{l}\sum_{A}^{B}(P_2 + P_1) - \sum_{A}^{C}(P_2 + P_1) - \sum_{A}^{C}(P_2 - P_1)\,\frac{a_1}{l} - \sum_{C}^{B}(P_2 - P_1)\,\frac{b_1}{l},$$

or,

$$h\,\frac{\delta\,T}{\delta\,x} = c_a\left[\frac{L}{l} - \frac{C_a + C_1}{c_a}\right] \qquad (13_2)$$

We find that essentially the same result can be obtained from equation (15) which refers to $C'\,K'$ or to its corresponding bar in the lower chord, viz.: that one extending from K towards the right. The equation we then find puts k_a for c_a, K for C, a_2 and b_2 for a_1 and b_1, and exchanges P_1 and P_2 with each other, as should be done in applying (13_2), an expression referring to $C\,K$, to the next panel at the right of $C\,K$.

Now, let some wheel W_u pass the point C from right to left, then is C_a increased by the amount W_u and C_1 by the amount $W_u\,\frac{a_1 + b_1}{l}$, both which quantities are positive though a_1 is negative. Hence T may reach a maximum as W_u passes C. In the graphical construction, similar to Fig. 10, let $k_u\,h_u$ be the vertical part of the weight line representing W_u, prolong $k_u\,h_u$ upward to a height

$$h_u\,n_u = W_u\,\frac{a_1 + b_1}{l}.$$

Then if the slope line $L \div l$ cuts $k_u\,n_u$, the wheel W_u at C produces a

Digitized by Google

maximum stress in the bar $C\,K$, for $k_u\,n_u$ is the amount by which $C_a + C_1$ changes as W_u passes C.

Again, whenever any wheel W_u passes from right to left of any joint of the same component system as C at the right of C, we find that C_1 is increased by the amount

$$2\,W_u\frac{p}{l}$$

by reason of the transference of W_u from P_1 to P_2 and the simultaneous sudden increase in the co-ordinate b_1 by the amount $2\,p$. Hence a maximum value of T may occur when any wheel passes any joint at the right of C and belonging to the same system as C, as it does not when any wheel passes any other joint at the right of C, including K.

Again, it appears in like manner that when W_u passes any joint of system I at the left of C, the increment of C_1 is the positive quantity

$$-2\,W_u\frac{a_1}{l}.$$

Hence the maximum stress in $C\,K$ cannot occur as any wheel passes any joint of the same system as C at the left of C, as it may when it passes any other joint at the left of C.

The nearer these last joints lie to $C\ K$ the larger are these increments, and consequently the more likely are they to produce maximum stresses in $C\,K$.

2°. In order to find the train position for a maximum stress in $C\,K$ when K lies in the right half of the span, let the inclinations of the web members shown in Fig. 7 be altered so that $C\,H'$ and $K\ C'$ are vertical, etc.

Now, apply the general equation (14) to this case, and after we have replaced the limit C' by K and put $c_a' = k_a$, and noticed that the third term vanishes as before, we obtain at once

$$h\,\frac{\delta\,T}{\delta\,x} = \frac{k_a}{l}\sum_{A}^{B}(P_2 + P_1) - \sum_{A}^{K}(P_2 + P_1) - \sum_{A}^{K}(P_2 - P_1)\frac{a_2}{l}$$

$$-\sum_{K}^{B}(P_2 - P_1)\frac{b_2}{l}$$

$$\text{or,}\qquad h\,\frac{\delta\,T}{\delta\,x} = k_a\left[\frac{L}{l} - \frac{K_a + K_2}{k_a}\right] \ldots\ldots\ldots\ldots (14_2)$$

We obtain a result which is essentially the same as this in applying equation (16) to find the position for a maximum stress in $C'\,K'$ equal to that in $H\,C$, with such changes of notation as fit it to $H\,C$ instead of $C\,K$.

Digitized by Google

It appears from equation (14_2) by making use of considerations precisely like those employed in connection with (13_2) that a maximum may occur in $C\,K$ in the right half of $A\,B$, when a wheel passes K.

It also may occur when any wheel W_u passes any joint of the same system as K at the left of K, the increment of K_2 then being

$$2\,W_u\frac{p}{l},$$

and finally it may occur when any wheel W_u passes any joint not of the same system as K, situated at the right of K, the increment then being

$$2\,W_u\frac{b_2}{l}.$$

These statements as to the positions which may cause maximum stresses in $C\,K$ in the right half of the span could have been made by symmetry from those already made respecting the left half of the span.

The various practical devices to be employed in the graphical application of equations (13_2) and (14_2) have been already sufficiently explained in connection with the discussion of equation (13_1).

Digitized by Google

LOUISVILLE CEMENT.

The undersigned is General Agent for the following Standard Brands of Louisville Cement:

FALLS MILLS (J. Hulme Brand),
BLACK DIAMOND MILLS (River),
SPEED MILLS,
FALLS CITY MILLS,
QUEEN CITY MILLS,
ACORN MILLS,
BLACK DIAMOND MILLS (Railroad),
FERN LEAF MILLS,
EAGLE MILLS.

This Cement has been in general use throughout the West and South since 1830; most of the public works having been constructed with it. Orders for shipment to any part of the country, by rail or water, will receive prompt and careful attention.

Sales for 1888, 1,202,118 Barrels.
Sales for 1889, 1,338,464 Barrels.

WESTERN CEMENT ASSOCIATION,
247 W. Main St., Louisville, Ky.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

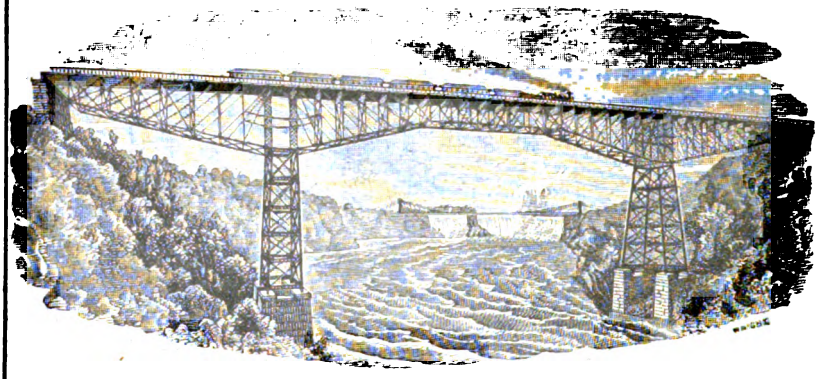
SELECT ADVERTISEMENTS WILL BE RECEIVED AT THE FOLLOWING RATES:

	TWELVE MONTHS.	SIX MONTHS.	THREE MONTHS.	ONE MONTH.
One Page.....	$100.00	$55.00	$30.00	$12.00
One-half Page.....	55.00	30.00	18.00	7.00
One-quarter Page	30.00	18.00	10.00	5.00
One-twelfth "Card".	12.00	.....	.....	....

Address the Secretary of the Society, 127 East 23d Street, New York.

Digitized by Google

UNION BRIDGE CO.



C. MACDONALD, GEO. S. FIELD,

C. S. MAURICE, EDMUND HAYES,

CIVIL ENGINEERS,

And Constructors of Iron and Steel Bridges, Viaducts, Roofs, Elevated Railroads, Marine Piers, etc.

Designs and Estimates will be sent on application to

General Office, WASHINGTON BUILDING,

1 BROADWAY, NEW YORK.

Works: { Athens, Pa. - - Capacity, 15,000 Tons.
Buffalo, N. Y. - - " 15,000 "

Digitized by Google

VIII 570

JUNE.

1890.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Library.

CONTENTS.

TRANSACTIONS. VOL. XXII.

No.		Page
438.	Address at the Annual Convention, at Cresson, Pa., June 26th, 1890. WILLIAM P. SHINN	359
439.	A Method of Taking Cross-Sections in Deep Rock Cuts by Triangulation. F. W. WATKINS	386
440.	Calculations of the Mean Horse-Power of a Variable Stream and the Cost of Replacing the Power Lost by a Partial Diversion of the Flow. WILLIAM H. GRANT	389
....	Discussion on Paper No. 440— By CHARLES E. EMERY, WILLIAM E. WORTHEN, EDWARD P. NORTH, JOSEPH P. DAVIS and WILLIAM H. GRANT	402
441.	Observations on the Forth Bridge. CHARLES E. EMERY	409

ILLUSTRATIONS.

LXIV.	Cross-Section, West Approach Cut, Weehawken Tunnel	388
LXV.	Geometrical Solution of Triangulation	388
LXVI.	Diagram A, Illustrating the Mean Yearly Flow and Horse-Power of the Bronx River	408
LXVII.	Diagram B, Illustrating the Mean Yearly Flow and Horse-Power of the Bronx River after the Diversion of 45 per cent. of the Normal Flow	408
LXVIII.	The Forth Bridge; Elevation and Sections	424
LXIX.	Diagrams showing Temperature Movements	424

The Transactions of the Society will be sent to any subscriber at the rate of $10 per year; and to clubs of ten or more, when ordered through the Secretary of an Engineering or Technical Society or Club who will be responsible for the payment, at 25 per cent. discount.

PUBLISHED AT THE HOUSE OF THE SOCIETY, 127 EAST TWENTY-THIRD STREET, NEW YORK, ON THE SECOND WEDNESDAY OF THE MONTH.

Entered according to Act of Congress, in the year 1890, by the American Society of Civil Engineers, in the Office of the Librarian of Congress, at Washington.

Digitized by Google

American Society of Civil Engineers.

OFFICERS FOR 1890.

WILLIAM P. SHINN, *President.*

ALPHONSE FTELEY, MENDES COHEN, *Vice-Presidents.*

JOHN BOGART, *Secretary and Librarian.*

GEORGE S. GREENE, JR., *Treasurer.*

CHARLES B. BRUSH, THEODORE VOORHEES, ROBERT VAN BUREN, WILLIAM LUDLOW, WILLIAM G. CURTIS, *Directors.*

Standing Committees.

ON FINANCE:—Charles B. Brush, Mendes Cohen, Robert Van Buren.
ON LIBRARY:—Alphonse Fteley, Theodore Voorhees, William Ludlow.

Special Committees.

THE PRESIDENT OF THE SOCIETY IS HONORARY MEMBER OF ALL COMMITTEES.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore M. Ely, J. M. Toucey, T. Egleston, Fred. Brooks.

ON THE COMPRESSIVE STRENGTH OF CEMENTS AND THE COMPRESSION OF MORTARS AND SETTLEMENT OF MASONRY:—F. Collingwood, D. J. Whittemore, W. W. Maclay, T. C. McCollom, George F. Swain, A. V. Abbott.

ON UNIFORM METHODS OF TESTING MATERIALS USED IN METALLIC STRUCTURES, AND ON REQUIREMENTS FOR THESE MATERIALS TO FURTHER IMPROVE THE GRADE OF SUCH STRUCTURES:—G. Bouscaren, William H. Burr, J. G. Dagron, R. W. Hunt, H. G. Morse, P. Roberts, Jr., H. B. Seaman.

MEETINGS.

Regular Meetings, on the first and third Wednesdays of the month, at 20 o'clock, except during July and August.

The rooms of the Society are open from 9 to 17 o'clock each business day, except Saturday, when they are closed at 15 o'clock.

The Library and Conversation Room will also, for the present, be open every *Wednesday Evening*, from 19½ to 22 o'clock. Members are invited to avail themselves of the opportunities afforded on these evenings, both for consultation of books and periodicals, and for conversation.

HOUSE OF THE SOCIETY—127 EAST TWENTY-THIRD STREET, NEW YORK.

Digitized by Google

SEP 10 1890

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

438.

(Vol. XXII.—June, 1890.)

ADDRESS AT THE ANNUAL CONVENTION AT CRESSON, PA., JUNE 26TH, 1890.

By WILLIAM P. SHINN, President, Am. Soc. C. E.

By a constitutional mandate it is made the duty of the President of the Society to deliver an address, which shall set forth the progress made in engineering science during the preceding year. Literal compliance with this provision necessarily involves reference to much with which you are more or less familiar. At the risk, therefore, of telling you of some things which you know much better than I do, I will endeavor, as best I can, to comply with the provision of our fundamental law. If, in doing so, I shall touch upon some facts which do not indicate progress, it may suggest to some of your minds how progress may be made.

BRIDGES.

Probably the most important engineering structure which has been completed since the last Convention is the Forth Bridge; notable because of its containing the longest spans in the world.

Digitized by Google

It is well known probably to most of the members that this bridge has two channel spans, each 1 710 feet inthe clear, with a clear headway of 150 feet under the bridge. These great spans are made up of two 680 feet cantilevers and a suspended span 350 feet long. The cantilevers are 340 feet deep over the piers and 40 feet at the ends, and the bottom chords are tubes tapering from the diameter of 12 feet to 5 feet, spaced 120 feet apart at the springing line and 31½ feet apart at the top. It is recorded of its construction that in the south span, which was connected up October 10th, 1889, the two arms met on the same level, but as the temperature was not as high as was expected, the rivet holes in the chords and connecting plates were ½ inch apart on the west side (the side exposed to the sun), and 1 inch apart on the other; the holes in the western side were brought over by a hydraulic jack, while on the east side the pull of 170 tons failed to bring them nearer than ½ of an inch. The girder was then heated by placing waste soaked in naphtha in the trough of the girder and firing it, which expedient was successful.

The test of these great spans, made January 24th, 1890, by placing on each of them the enormous weight of 1 800 tons, showed deflections in exact accordance with the calculations of the engineers, while on January 19th, with a registered wind pressure of 37 pounds per square foot the lateral movement of the cantilever was less than 1 inch. The bridge was opened on Tuesday, March 4th, 1890, with great ceremony, as was its due. The total amount of steel used in the bridge proper and the approach viaduct is reported as 59 500 tons, and the total cost as about $15 500 000. This is truly a triumph of engineering.

The Merchants' Bridge, St. Louis, which was completed on February 28th, 1890, consists of three trusses of about 525 feet each, planned to give openings of not less than 500 feet each in the clear, and a clear headway in the center of 52 feet above high water. The approach spans on either side make the total length of the permanent steel and masonry bridge about 2 420 feet. This bridge is under the charge of Mr. E. L. Corthell, Member of the Society, as Chief Engineer, and Mr. George S. Morison, Member of the Society, as Consulting Engineer.

The Kentucky River Bridge, which was completed in August, 1889, is a cantilever, and is of interest both for the length of span and (considering the magnitude of the work) the rapidity with which it was

Digitized by Google

constructed. The bridge consists of two cantilevers of trusses 60 feet deep at the point of support and 20 feet deep at the ends of the arms, supported by two steel towers 135 feet high and 551 feet apart; the total length of the structure, including approaches, is 1 658 feet. The bridge was completed, ready for the passage of trains, in six months from the date of beginning work on the foundations.

The bridge over the Thames River, in Connecticut, constructed under the supervision of Mr. Alfred P. Boller, Member of the Society, is remarkable mainly for having the longest draw span in the world. The entire length of the bridge is 1 423 feet, consisting of two spans of 150 feet each, two 310 feet each, and a draw span of 503 feet, affording two clear passage ways of 225 feet each for vessels; the entire superstructure is steel, and it was opened for traffic on October 10th, 1889.

The Coteau Bridge over the St. Lawrence River was opened for traffic early in April, 1890. The extreme length of the bridge, including the islands which are trestled, is 6 035 feet; the bridge is of the double triangular riveted lattice design. The north channel has one span of 139 feet, two spans 175 feet each, and a draw span of 355 feet. The middle channel is covered by ten spans of 217 feet each, and the south channel by four spans of 223 feet each. On account of the swiftness of the current the caissons were built ready for lowering and floated into position suspended between two floats; the trusses were erected on the south shore in 6 feet of water, on false work of the same elevation as the bridge, and the false works and trestles were then lifted on floats on scows, which were towed to the bridge site and placed in position between the piers, so that by sinking the scows the span was lowered to its permanent seat, after the plan adopted on the Hawksbury Bridge.

Bridges Under Construction.

The new Louisville and Jeffersonville Bridge is well under way; this bridge will surpass all previous records in length of truss spans; it will have two 550 feet, one 553 feet and two 431 feet spans, all center to center of piers; the total length of the bridge, including iron viaduct approaches, will be 8 505 feet.

The bridge over the Ohio River at Brunot's Island, three miles below Pittsburgh, which is being constructed under the supervision of Mr. M. J. Becker, Past President of the Society, will have a channel span of 500 feet, which is to be erected under the direction of Mr.

Digitized by Google

Charles L. Strobel, Member of the Society, and will be notable from the fact that the channel span is to be erected on false works set upon piles contiguous to, but at right angles to the line of the bridge, and after erection the span with the false works is to be transferred to coal boats (used as pontoons), to be floated into position, as was done in the case of the Hawksbury Bridge.

The bridge over the Colorado River at Red Rock, a few miles below the "Needles," which is to furnish the crossing for the California and Arizona Railway, is well under way. Its main span is a cantilever, 660 feet between centers of piers, consisting of cantilever arms 165 feet each, and a suspended span of 330 feet. The plan was made by Mr. J. A. L. Waddell, M. Am. Soc. C. E.

The bridge over the Mississippi River at Memphis is also well under way; the span between piers one and two will be 791 feet, then come two spans of 621 feet each, and a suspended span of 350 feet, while there will be nearly one mile of 150 feet spans; the clear headway under the bridge will be 75 feet, and the estimated cost is $3 000 000.

The Tower Bridge now under construction across the Thames at London, is remarkable for the peculiarities in its design. It consists of two short spans, each 270 feet, and a middle span of 200 feet, resting, when in place, 29 feet 6 inches above high water, but divided in the center and hinged at the ends, so that its two leaves may be raised to a perpendicular position, leaving an opening to the truss connecting the two towers at a height of 135 feet above water level.

Bridges Proposed.

The North River Bridge, when constructed, with its span of 3 000 feet, will restore to this country the prestige which it enjoyed at the time of the completion of the East River Bridge, of having the longest span bridge in the world. It seems to be fairly in the way of accomplishment. The bill authorizing its construction passed the House of Representatives at Washington, on April 2d, and provided that the bridge shall have not less than six railroad tracks, with a capacity for four additional tracks for future enlargement; that it shall be constructed with a single span over the entire river between the towers located between the shore and the pier head lines in each State, and have an elevation not less than that of the existing Brooklyn Bridge. Construction of the bridge shall be begun within three years, and com-

Digitized by Google

pleted within ten years after beginning. All persons, railroad and telegraph companies, shall be entitled to equal rights and privileges for the use of same for a reasonable compensation.

One of the most important bridges projected, and upon the approaches to which much work has been done, is the bridge over the Hudson River at Anthony's Nose, known familiarly as the Peekskill Bridge. It is intended to unite the New York and New England Railroad with the railroads west of the Hudson. It will be a double-track suspension bridge of 1 600 feet span, clearing the whole waterway at the narrowest point in the Hudson below Albany, and it will be 190 feet above the water.

The proposed bridge over the Mississippi River, about 9 or 10 miles above New Orleans, designed by Mr. E. L. Corthell, Member of the Society, if constructed, will have three discontinuous spans about 866 feet each in length, and a clear height at the center span above extreme high water of not less than 82 feet. The estimated cost will be about $3 000 000.

A bill has been introduced into Congress authorizing the construction of a bridge over the Delaware River, uniting Philadelphia and Camden. It is designed to be a suspension structure 4 440 feet long, 135 feet high, and if constructed will accommodate railway, carriage and pedestrian travel.

The most pretentious project is that of the proposed bridge over the English Channel, plans for which have been brought forward in elaborate design, by Messrs. Snyder & Company, and Hersent, with Sir John Fowler and Mr. Benjamin Baker as consulting engineers. This plan proposes a bridge 24½ miles in length, with one hundred and eighteen piers and two abutment piers. The spans vary in length from 250 meters (820 feet) to 500 meters (1 640 feet). The general design calls for a short span of 250 to 300 meters between each two cantilever spans of 425 to 500 meters, so that the short and long spans alternate across the channel; the greatest depth of water is 180 feet, and the clear headway under the trusses would be 177 feet at high water; the piers are proposed to be of masonry to a height of 21 meters above high water, surmounted by two steel cylindrical piers varying in height between 131 and 140 feet to the seat of the truss. The total weight of the whole bridge is estimated at 771 265 tons, and the total cost $172 000 000.

Digitized by Google

Bridge Floors.

The accident known as the St. George Bridge Disaster, which happened on the evening of February 27th, 1889, is a case in point of the inefficiency of bridge floors. This bridge consisted of eight spans of 50 feet each, composed of iron trusses upon which cross-ties 8 x 12 inches were laid, at least 8 inches apart, and upon these ties the rails were spiked. At a point about 450 feet from the end of the bridge, the engineer noticed that the driver tire was broken; this resulted in the breaking of a side rod, so that upon entering on the bridge the whole train was derailed. There was a heavy wooden guard-rail connecting the outer ends of the cross-ties, but there were no guard-rails proper close to the rails and no re-railing guards at the end of the bridge; the result was that the ties bunched together, leaving gaps which the wheels could not jump, thereby destroying one span of the bridge and causing two cars of the train to fall to the ground, and resulting in thirteen deaths and twenty-nine severe injuries and a money loss of over $200 000.

On August 22d, 1889, there occurred on the Knoxville, Cumberland Gap and Louisville Railroad at Flat Creek, a somewhat similar accident known as the Flat Creek disaster; two passenger cars were derailed in approaching Flat Creek trestle. It had a flooring of ordinary cross-ties without any guard-rails whatever; the ties were bunched, resulting in one passenger car falling to the ground a vertical distance of about 18 feet below the track, and out of the thirty-four passengers in that car, twenty-nine were killed or seriously injured.

In both of these cases had there been proper guards in the bridge approaches and proper guard-rails on the bridge, and had the cross-ties been laid not more than 3 to 4 inches apart with spacing blocks between them to prevent bunching, it is not probable that either disaster would have occurred.

The subject of bridge floors is one which has long needed more careful attention, and it is gratifying to observe that the New York Central and Hudson River Railway has awakened to its importance and is now building or refitting many of its short-span bridges with solid buckle-plate, or plate-iron floors, upon which the ballast is carried uninterruptedly. Every experienced engineer of maintenance of way knows how extremely difficult it is to secure and maintain such perfection of surface that the transitions from ballast to track on wooden

Digitized by Google

bridge floors, and *vice versa*, shall be smooth and without shock. This difficulty is avoided by the character of floor adopted by the New York Central. Its advantage in case of derailment was shown on November 12th, 1889, when five cars of a west-bound freight train left the rails and dragged across the new Erie Canal bridge at Rome, which had been provided with a floor of this character. The cars were scattered all over, two having their trucks pulled from under them, the ties and road-bed were torn up so that it looked as if the company had been trying to run one train on two tracks, but no damage was done to the bridge.

It is plain that true economy calls for construction of this character where it is possible, or where it is not found practicable, the replacing of iron bridges with stone arches, as is being done by the Pennsylvania Railroad Company.

The most pernicious system in use is probably that in which the rails are spiked directly upon the stringers without the intervention of cross-ties. There are probably many hundreds, and it is possible, thousands of such bridges in this country, and every one of them, no matter how short the span, is a serious cause of danger.

Railroads.

According to "Poor's Manual," the mileage of main track in the railroads of the United States at the end of 1888 was 156 082. According to the *Railroad Gazette* the new mileage constructed during 1889 was 5 300, which would make the mileage, at the end of 1889, 161 382.

According to "Poor's Record," the quantity of rolling stock of the railroads in the United States at the end of 1888 was as follows: Locomotive engines, 29 398; passenger cars, 21 425; baggage, mail and express cars, 6 827; freight cars, 1 005 116.

The *Railroad Gazette* reports freight cars as having been built by thirty-six car building companies in 1889, 70 546; as this is an addition to the cars built by railroad companies it is probable that at the close of 1889 there were not less than 1 100 000 freight cars on the railroads of this country. It is greatly to be regretted that none of the statistics published give the car mileage of all the railroads of the United States; but there is reason to believe that while commendable progress has been made in the tonnage or capacity of freight cars and in the number of cars and weight of loading hauled by engines, the average move-

Digitized by Google

ment of freight cars still remains in the vicinity of 25 miles per day, or about what a car when in motion should average in two and a half hours, so that on the average the freight cars of the United States stand still during nine days for every day that they are in motion. As cars are constructed with the prime object of using them in transporting freight from one place to another, it is not creditable to railroad managers that nine-tenths of their time should be devoted to the other uses of standing idle, receiving loads and holding them in store. The efforts of the writer while in charge of the management of the New York and New England Railroad showed conclusively that, with a proper system of demurrage charges for delays in loading and unloading cars, their average movement could be increased more than 50 per cent., and that, too, without any serious inconvenience to the patrons of the road or any practical injury whatever to their business.

A comparison of the number of tons of freight moved in 1888 with that moved in 1882, shows an increase of about 63 per cent., while the increase in the number of cars from the end of 1881 to the end of 1887 was about 48 per cent.

Considering that during that period the average capacity of freight cars has been increased by at least one-half, it does not show that any improvement has been made in the use of the cars during that time. In the economy of railroad management in other respects very satisfactory improvement is evident. Compound locomotives are receiving a fair trial in this country. The Pennsylvania Railroad Company brought over from England a compound locomotive weighing 95 200 pounds, with 6 feet 3 inch drivers; two 14 x 24 and one 30 x 24-inch cylinders. It is reported that the engine appeared to save 8 pounds per mile of coal, but the difficulty in starting the train was too serious to be overlooked.

The Baltimore and Ohio Railroad Company had a four-cylinder compound locomotive constructed by the Baldwin Locomotive Works, the cylinders being respectively 12 x 24 and 20 x 24, driving wheels 68 inches in diameter; these cylinders are placed one over the other, both piston rods being connected to the same cross-head. I apprehend that this form of construction will be found more satisfactory than that of the English engine.

The Michigan Central Railroad Company had a compound engine of the ten-wheel type constructed by the Schenectady Locomotive Works.

Digitized by Google

This engine has but two cylinders, one on each side, the high pressure being 20 x 24 inches, the low pressure 29 x 24 inches. Weight of the engine was 126 800 pounds, of which 97 000 was on driving wheels. In a train of 359 tons, including the weight of the engine and tender, the engine ran 228 miles with a consumption of a little over 7 tons of coal, the standard engines using 7 tons for 152 miles. This seems to have been a very satisfactory performance. It is reported that the East Tennessee, Virginia and Georgia Railroad has given an order to the Schenectady Locomotive Works for three compound engines on the Mallett system, same as the Michigan Central.

The Strong locomotive has given promise of economical results; the second engine of this character, named the A. G. Darwin, has cylinders 19 x 24 inches, four driving wheels 68 inches in diameter and weighs 136 000 pounds with 76 000 pounds on the drivers. The engine is reported as having made a run on the New York, Lake Erie and Western Railroad between Jersey City and Buffalo with the day express train starting with six cars from Jersey City and arriving at Buffalo with eight; there is no record of the consumption of coal, the main feature of the run being that it is usually run by four engines, and in this case was made by one.

In a report published in the *Cincinnati Commercial Gazette* in April, 1890, of the use of the Strong locomotive on the Cincinnati, Hamilton and Dayton Railroad, the General Superintendent states: "I have ridden on the Strong engine, pulling seven cars at the rate of 1 mile in forty-two seconds for some distance on an upward gradient of 10 to 15 feet per mile, and on a practically level or slightly down grade a mile was made in forty seconds. When she ran up hill, 8 miles, with nine heavy passenger cars, making fair schedule time, she arrrived at the top blowing off steam, and carrying a pressure of 170 pounds; as to coal consumption, the engine shows as nearly as we can determine by the ordinary appliances about one-third less coal than an ordinary engine doing the same work." This is very high praise.

It is claimed that material advantage in the increase of locomotive traction has been derived from the use of an electrical apparatus causing increased adhesion of the wheels to the rail. The use of polygonal wheels, the tires of which were ground into facets of about five-eighths of an inch face, has resulted in total failure, as might well have been expected.

Digitized by Google

It is in matters of safety appliances that the greatest progress has been made during the past year. The subject of car heating has been taken up by the Legislatures of a number of the States, notably Massachusetts, Connecticut, New York and Iowa, in each of which the Legislature has fixed a definite time within which all cars must be heated by steam.

It is stated in a recent number of *Engineering News* that during the winter of 1889 and 1890 there were heated by steam from the locomotives all passenger trains on 8 465 miles, over half the cars on 10 912 miles and from 10 to 50 per cent. on 17 516 miles, while roads having 10 per cent. or less of their cars so heated aggregated 50 550 miles. In this commendable advance, New York and the New England States are decidedly in the lead, there being on New England railways 1 420 cars equipped for steam heating out of 2 704 in use, and on New York railways 3 806 equipped for steam heating out of 4 163 in use.

Experiments conducted on the New York and New England Railway while under the writer's supervision in the winter of 1888–89, show that the heating of passenger trains by steam from the locomotives was not only practicable but economical. The occasional car stove accidents, such as occurred in the last week of January on the Louisville, New Albany and Chicago Railroad, with another on the Cincinnati, Hamilton and Dayton Railroad, and still another on the Denver and Rio Grande, all in the same week, emphasize the importance of this improvement proceeding rapidly.

In lighting cars still more progress has been made, the kerosene lamp giving place to compressed gas. Of the systems presented for competition in car lighting the Pintsch system of lighting with compressed gas made from oil appears to have a fairly-earned lead.

It is reported that in Europe there are over 30 000 passenger cars now equipped for and using Pintsch gas, and that arrangements have been made for its use upon an aggregate of over 35 000 miles of railroad in this country during the present year.

In power brakes for freight cars a great advance has been made; the Westinghouse Air Brake Company reports that its automatic brake was placed during 1888 on 27 696 freight cars and during 1889 on 24 036, and that at the close of 1889 there were 101 576 freight cars equipped with it, or nearly 10 per cent. of the whole number of freight cars in the country.

Digitized by Google

In 1888, after a somewhat exhaustive series of experiments by a committee, of which the writer was chairman, had demonstrated the actual superiority of vertical hook couplers over all others, the Master Car Builders adopted as their standard the vertical hook coupler of the Janney type, and several of the States (particularly Massachusetts, New York and Iowa) have passed laws requiring automatic couplers to be put on all newly-constructed freight cars, and within a certain period they must be applied to all the freight equipment used by roads within these States.

A statement in the *Engineering News* of December 21st, 1889, shows freight cars equipped with the Master Car Builders' standard coupler 56 050; with other automatic couplers, 15 755; making a total of 71 805 cars equipped with automatic couplers. The result of the experiments with the Master Car Builders' type of coupler on the Atchison, Topeka and Santa Fé Railroad has shown that it will, by reason of its close coupling and automatic action, more than pay for itself in the decrease of losses resulting from breakages of trains and damages to employees, and that it is especially adapted to the fast freight service.

The record of the Janney couplers, as given in the *Engineering News* of February 15th, 1890, appears to show that they stand the shocks of service quite as well as couplers of any other type.

As the organizations of trainmen are resolving in favor of automatic couplers, it appears safe to say that within the next decade all freight cars will be so equipped. In this connection also it is well to refer to the Westinghouse friction buffer as an additional safeguard. A test made on Tuesday, February 25th, 1890, in the West Shore yards, New York City, with twenty-two Pennsylvania Railroad cars of 60 000 pounds capacity each, all fully loaded and equipped with Janney couplers, Westinghouse quick acting brakes and Westinghouse friction buffers showed many remarkable results. It is recorded that a ten-wheel engine attached to eight cars of the train was moved off a little distance, the brakes being set upon the remaining cars. The front portion of the train was then moved down upon the rear part at the speed of 5 to 7 miles per hour, but so much of the shock was taken by the buffers that no damage was sustained in any part. It seems clear that with automatic brakes, automatic couplers and such buffers it will be possible and safe to very materially increase (perhaps to double) the average speed of freight trains.

Digitized by Google

The subject of safety signals is one of growing importance; the leading trunk line roads have long since adopted the block system; but such accidents as that of the Pennsylvania Railroad Company on the New Jersey meadows some years ago, show that something else is needed, and automatic electric signals appear to answer the purpose.

The writer had three years' knowledge of their use on 12 miles of the road under his charge without a recorded failure, the signals being then about a mile apart, but owing to the frequency with which trains were run they have recently been doubled, placing them half a mile apart. A class of signals which should come into general use, even upon roads running comparatively few trains, is a mechanical signal which can be set by the switchman or any station man at a sufficient distance back of each switch and station as to enable an approaching train to stop before reaching it; such a signal to be thrown to danger whenever a train is taking siding or standing at a station.

A very large proportion of the collisions lately recorded have been due to trains taking siding or standing at stations being run into by following or approaching trains.

The railway accident statistics are not only long but ghastly, and everything possible should be done to reduce their number.

A record compiled by the *Engineering News*, in its issue of June 7th, 1890, shows for the year ending June 30th, 1889:

	Killed.	Injured.
Passengers	310	2 146
Employees	1 972	20 028
Other persons	3 541	4 135
Total	5 823	26 309

As one-third of the killed and nearly four-fifths of the injured are employees, it is safe to say that by the adoption of the safety appliances hereinbefore mentioned, the number of persons killed and injured can be reduced in the aggregate by at least one-half; it is true that by far the largest proportion of the killed, say 3 541, are persons other than employees and passengers, of which 2 215 are probably trespassers.

A comparison of the railroad accidents in the United States with those in Great Britain shows that while there was one passenger killed in the United States for every 39 601 458 miles traveled, and in Great Britain for every 55 538 688 miles traveled, there were passengers injured in the United States one for every 5 720 624 miles traveled, and in Great

Digitized by Google

Britain for every 4 220 616 miles traveled, so that the difference in favor of safety of travel in Great Britain is not as material as has generally been supposed. This is more effectively shown by the following comparison of passenger casualties per million passenger train-miles:

	Killed.	Injured.
United States	1.04	7.2
Great Britain	0.70	10.4

One of the most important points to which I would call the attention of railway managers is the importance of keeping trains on time. This statement seems trite, but that it is a live issue is evidenced by an editorial in a leading Southern paper which recently adopted the plan of publishing daily a list of trains running into the city which arrived behind the schedule time. I cannot do better than quote: "The duty of the railways to the traveling public is clear; it is simply to deliver passengers promptly and surely on the advertised time; the speed of a train, the number of trains, these are matters of convenience and are regulated by the desire to get travel, but to put a passenger where you advertise and put him exactly where you advertise, when you advertise to put him there, is a matter of contract and it rests on a solemn obligation. This contract the company is bound to keep; an accident may prevent it once in a year or once in a month and the road be held blameless, but when there is frequent and avoidable failure to keep this contract with the public, the railroad deliberately fails in its duty. If the advertised schedule is a faster one than can be regularly and reasonably run, it should be changed. If more trains are dispatched than can be promptly delivered, the number should be reduced. The public may criticize slow or infrequent trains, but if the road keeps its advertised promise of delivery its integrity can be maintained and it cannot be hurt. The public, too, had rather travel in ordinary trains and 'get there' than in 'fast mails' that are slow, or in 'cannon balls' that go astray, or 'through limiteds' that find their limit before they get there."

These remarks are well timed and very much needed. It should be impressed upon our friends in charge of railway management that it is far more important for the trains to be prompt than for them to be fast; and I regret to say that from observation and experience it is my conviction that regularity and promptness of trains upon many of our leading railway lines is not as satisfactory to-day as it was twenty

Digitized by Google

years ago. This is partly due to the high speed schedules and heavy trains, but (as was remarked in the newspaper article referred to) the important thing is that the train should get there at the time called for in the schedule, and there is a certain way of its accomplishment—if the train from any cause cannot make the schedule time the schedule time should be made to suit the train.

Two Americas Railway Project.

A most notable project in the line of railway construction is the one which is known as the "Two Americas Railway," designed to connect North and South America, from the United States on the north to Chili and the Argentine Republic on the south. The construction of this railway having been recommended by the recent Pan-American Congress, the President has transmitted to Congress a report of the Secretary of State, recommending the appropriation of $60 000 as the proportion of this country for the cost of the survey. Chimerical as it may seem, it is probable that at no distant day the construction of this railway will be commenced.

Ship Railways.

It is to be regretted that the unfortunate death of our esteemed Past Vice-President, James B. Eads, appears to have caused his latest project, the Tehuantepec Ship Railway, to be, at least for a time, abandoned. It is interesting, however, to notice that there is at this time under construction a ship railway which, in many respects, will fully demonstrate the correctness of Captain Eads' plan. The Chignecto Ship Railway now under construction across Nova Scotia will connect the Bay of Fundy with the Baie Yerte of the Gulf of St. Lawrence. The length of the road is about 17¼ miles. The road-bed will consist of two parallel tracks 18 feet between centers and laid to the usual 4 feet 8½-inch gauge. The rail will be 110 pound section, 6¼ inches high with 6¼-inch base. The company is fortunate in having been able to determine upon an absolute straight line for the whole length of the road, thus avoiding the difficulty which Captain Eads was obliged to provide for, of turning material angles; the lifting docks are designed for vessels of 1 000 tons or less capacity.

It was reported last month that 14 miles of the road-bed had been graded; all the rails being delivered, most of the hydraulic machinery is on the ground, and the masonry of the lifting docks is well advanced. It is expected that the railway will be finished next year.

Digitized by Google

Ship Canals.

Since the abandonment of the work on the ill-advised Panama Canal, there has been little advance to chronicle in this branch of engineering. It is, however, gratifying to be able to state that while the foreign enterprise referred to appears to have definitely failed, the American Nicaragua Ship Canal appears to be making substantial progress. It was reported in September, 1889, that the work had been fairly begun, and the subsequent reports show that it is being conducted in a systematic way with a view to an economical carrying out of the enterprise. It is particularly interesting to the Society, from the fact that one of our Members, Mr. A. G. Menocal, is the chief engineer, and that a favorable report upon the practicability and cost of the canal has been made by a board consisting of five Members of the Society headed by our worthy Secretary.

New Croton Aqueduct.

The New Croton Aqueduct, in charge of Mr. Alphonse Fteley, Vice-President Am. Soc. C. E., is nearing completion, and the Chief Engineer promises that it shall carry water on July 15th prox.

This aqueduct is notable as being the longest tunnel constructed in modern times.

Eiffel Tower.

Among the unique and remarkable constructions of the past year was the Eiffel Tower, notable as being the highest structure of modern times, and probably the highest of which we have any recorded history. Its features are so well known that it is unnecessary to refer to them here.

Electrical Engineering.

In no other branch of the profession have as great strides been made as in that of electrical engineering. At the close of 1889 there were in the State of New York alone ninety-two electric light companies furnishing light to cities and the public. The price of gas, which in 1877 for two hundred and ninety cities averaged $3 15, had in 1887 fallen to $2 per thousand feet. I have not been able to procure an accurate record of the number of electric light stations, with the number of electric lamps in use in the United States, but one electrical company alone had on the 1st of June three hundred and one stations with a total generating capacity of 554 350 lights of 16-candle power each.

Digitized by Google

Of electrical street railways there were on May 1st, according to *Electrical Power*, two hundred and forty in operation and under construction.

When it is considered that the first of these dates back only to 1886, the showing is very remarkable. In July, 1889, there were reported in use seventy-two electrical railways with a total length of 400 miles of line.

One of the most remarkable advances in the use of electricity is that of electrical welding. From some experiments made at the station of the Malden Electric Company on March 7th, 1889, it was found that to weld a 2-inch bar of wrought-iron the entire current generated by a 16-horse-power engine occupied one minute and twenty seconds in heating the joint to the welding temperature. The weld was completed in five seconds, making one minute and twenty-five seconds in all.

One of the remarkable features of this method of welding appears to be, from a microscopic examination of a butt welded bar which had been etched with acid, that for about $\frac{5}{16}$ inches each way from the point of joining, the metal seems to have been refined and made more homogenious. From tests made upon the tensile strength it is reported that in the welding of the ordinary commercial metals a tensile strength at the weld of 90 per cent. of the strength of the unwelded metal is obtainable.

Water-Power.

It is somewhat remarkable that the use made of water-power at the present time is but a fraction of that which is easily available in many sections of the country. Even in the State of Connecticut, where water-power is availed of quite largely, it is probable that not one-fourth of the available water-power is at present in use. The greatest advance in this respect, which is now under consideration, is the utilization of the power of the Niagara River. A company has been formed for the purpose of constructing a feeding canal, taking the water from the Niagara River from a point about 1½ miles above the Falls, the wells to be located along the sides of the canal, and the tail water to be discharged into a tunnel which shall open into the Niagara River about a quarter of a mile below the falls. The projected works are estimated to be capable of developing about 120 000 horse-power, drawing from the river only four-tenths of 1 per cent. of the volume of the passing water. It is intended to utilize the water under a pressure as

Digitized by Google

high as 100 pounds per square inch. The plans for the tunnel and canal are nearly finished and contracts for their construction are soon to be let.

Sewage.

A branch of engineering of great and rapidly growing importance is that of providing for the sewage of large cities.

It is only within a comparatively few years that any efforts have been made to do otherwise with city sewage than to discharge it into the nearest running stream at the most convenient location. The result has been the serious contamination of the streams, in many cases affecting the water supply of cities lower down, and always with the result of having more or less offensiveness connected with the discharge of sewage. One of the most radical improvements which has been made in this regard was the construction of the Boston system of sewerage, with which most of you are familiar. There are so many growing cities in the interior of the country located on streams of insufficient volume to take the sewage without dangerous pollution that strenuous efforts are being made to utilize it as a fertilizer, or to reduce it to a condition of "innocuous desuetude" by filtration or by chemical, and, in some cases, electrical treatment. One of the most radical methods of disposing of sewage is to filter it in such manner as to take out all the organic matter, which is then cremated under such conditions that it furnishes the fuel for raising the steam to carry on the operation.

What is known as the Hardie system is a combination of precipitating reservoirs, pneumatic and electrolysis reservoirs and a cremating furnace.

The study of this subject is well worthy the attention of the profession, as there can be no more important questions, involving, as they do, the health of entire communities, than the problems of water supplies and sewers, and it is gratifying to be able to report that public interest is increasing in both branches of the subject.

Drainage.

One of the greatest undertakings of modern times is the drainage of the valley of the City of Mexico. The average altitude of the valley is 7 412 feet above sea-level, and its area 1 657 square miles. The total area in hydrographic surface of the valley, including the bottom and the mountain sides, which drain towards the interior, is about 2 711 square

Digitized by Google

miles; there are in the valley six lakes of considerable size, forming a crescent around the City of Mexico. The city stands upon the banks of Lake Texococo, which is the lowest in elevation and which is above the level of the city.

It is proposed to construct a drainage canal for the distance of 48 kilometers, or nearly 30 English miles, from the city to the mouth of the tunnel; this tunnel will extend through the hills to a distance of about 10 000 meters, or nearly 8 miles, in length, which, with a fall of one in one thousand, is to give a capacity of discharge of 61.70 cubic feet per second when full to the maximum height of flow at 13 feet above the bottom. Sixty-five per cent. of the sectional area being employed for flow of water, the velocity of flow is calculated at 7.54 feet per second.

The section of the tunnel is ovate, with a vertical diameter of 17.7 feet and a width at water level of 12.88 feet. The canal and tunnel were commenced on March 25th, 1889, and are to be finished by September 25th, 1891.

Common Roads.

Special and needed attention has of late been given to the subject of the improvement of our common roads, than which certainly no worse can be found in any civilized country.

In their Messages at the beginning of 1889 the Governors of both Massachusetts and Pennsylvania gave special attention to this subject, the former recommending particularly the passage of a law providing for a consulting engineer to give advice in regard to the improvement of the common roads of the State. The Engineers' Society of Western Pennsylvania took very prompt action in the matter by preparing and presenting to the Pennsylvania Legislature a bill having for its purpose improvement in the location, opening, vacation, construction and maintenance of highways, roads and bridges in the several counties of the Commonwealth; this draft, however, did not become a law.

The State of New Jersey did enact a very advanced law upon the subject, authorizing the employment of engineers and the improvement of country roads upon an intelligent basis.

The Governor of New York, in his Annual Message for 1890, recommended that the country roads of the State be built and maintained by the State, and that the work be done systematically under the supervision of the State Engineer. It is shown that good macadamized roads

Digitized by Google

16 feet wide cost only about $3 000 per mile, while Telford pavements with macadamized covering 16 feet wide cost about $8 000 per mile.

It is represented by Professor J. W. Jenks, in his paper upon "Legislation of the American States on Roads," based upon the conditions in Illinois, that a full load can be carried on the State roads only during three months in the year, two-thirds of a load during three months and half a load during six months. He claims that if the State had a good system of roads the farmers would be benefited to the extent of $116-000 000 in the value of their farms, and that they would save $15 300 000 annually in hauling, or 6 per cent. interest on $255 000 000. These are large figures.

An estimate made by Mr. Thomas H. Johnson, M. Am. Soc. C. E., of a committee of the Engineer's Society of Western Pennsylvania, was to the effect that the loss to the community of that State from bad roads would amount to at least $6 000 000 per annum, while there is at present a road tax collected of about $4 000 000 per annum, which is so expended as to be utterly wasted, which, added to the saving by good roads would represent an expenditure that might be made of $10 000 000 per annum.

There is no better example of the effect of good roads than is to be found in Union County, N. J., where the act of the Legislature of that State has been adopted more thoroughly than in any other portion of this State. The vicinity of Elizabeth, Westfield and Plainfield has a population which would not have been attracted there without good macadamized roads.

It is to be noted as significant steps in the right direction that the University of Pennsylvania has offered prizes of $400, $200 and $100 for the best three papers on road-making and maintenance, while the Case School of Applied Sciences of Cleveland offers free instruction in road-making in the form of three lectures. Probably there is no one subject in this country offering opportunities for greater improvement than that of common roads.

It is well known that during the past winter many sections of the country were paralyzed in their business of shipping agricultural products because, owing to the absence of severe frost, the roads were, for months, absolutely impassable.

Poetsch Freezing Process.

One of the most remarkable improvements in the driving of shafts is that of the Poetsch freezing process as applied to quicksands.

Digitized by Google

This process consists in sinking 8-inch wrought-iron pipes plugged at the bottom and connected with malleable iron couplings at the distance of 5 to 10 feet outside the circumference of the shaft to be excavated; inside of these large pipes is a 1¼-inch pipe terminating within 10 inches of the bottom; both series of pipes are connected with tubing of their respective diameters, which tubing leads to a couple of huge tanks in the freezing plant and engine house; the ice machine operates on a solution of chloride of calcium, reducing its temperature to about 17 degrees Fahr. below zero. This solution is forced into the series of small pipes, dropping to the base of the large pipes. The pressure forces the liquid up the large pipes and back into a tank from which it again passes to the ice machine. The result of the continued circulation of this solution is the absorption of the natural heat of the ground and the freezing of the earth around the pipes; these congealed circles gradually widen, forming a solid impervious wall within which the treacherous quicksand is held.

The process in this country is controlled by a company, at the head of which is General William Sooy Smith, Member of the Society, and the first work done in this country was the sinking of a shaft for the Chapin Iron Mining Company, at Iron Mountain, Michigan, the shaft being rectangular, 15½ x 16½ feet and about 100 feet deep. It is recorded that after forty days' freezing an ice wall 10 feet thick was formed around the shaft, and that in thirty days more it had reached the rock. It is further stated that upon reaching the rock, water was found to enter, which was shut off by a continuation of the freezing process.

An account is given of a shaft sunk in Belgium by this process. This shaft was sunk to a depth of 190 feet before the quicksand was struck, and was afterwards carried through 62 feet of quicksand by the use of this freezing process.

Metallurgy.

Great advances are being made in the domain of metallurgy. Within ten years the output of a blast furnace of 20 feet bosh has been increased from about 100 tons per twenty-four hours to 325 tons in twenty-four hours. This has been accomplished by the recognition of the fact that the smelting of ore is a chemical process, and that the amount of product which can be derived from a given plant is in direct proportion to the amount of oxygen which can be brought in contact with the carbon

Digitized by Google

of the fuel within a given time, under such conditions as to result in economical combustion.

The production of pig iron in the United States has increased from 2 301 215 gross tons in 1878 to 7 603 642 gross tons in 1889, an increase of 5 302 427 gross tons, or 230 per cent., in eleven years. The production of pig iron in Great Britain was 6 381 051 gross tons in 1878, and 8 245 336 gross tons in 1889, so that while the production in the United States was only 36 per cent. of the British production in 1878, it was 92 per cent. thereof in 1889.

One of the most interesting developments in metallurgical manufacturing is that of the spiral welded steel tubes now being manufactured at Orange, N. J., this process making it possible to manufacture from rectangular strips of metal of moderate widths tubes of almost any required size and strength, and it is claimed that experiments show that while tubes having a longitudinal seam are liable to open at the weld from pressure, the spiral welded tubes act somewhat on the principle of the arch, and the seam is compressed and compacted by the internal pressure. Another very important and most interesting development in metallurgy has been the improvement in the manufacture of aluminum, this heretofore very expensive metal having, by the latest process, been brought within the reach of manufacturers for many uses requiring comparatively cheap material, the Pittsburgh Reduction Company, which is probably the largest producer of the metal, furnishing 95 per cent. pure aluminum in quantities of 1 000 pounds and upwards at $2 per pound. This enables its use, heretofore confined to expensive instruments, to be extended to ordinary domestic and culinary vessels, for which its qualities of lightness in weight and non-oxidation make it particularly valuable. There is reason to believe that its cost will be so far reduced within a reasonable period as to enable it to be sold for less than $1 per pound.

Failure of Dams.

The site of South Fork Dam, the failure of which was the cause of what is well known to you all as the Johnstown disaster, is but 7 miles from our place of meeting, and those who are curious to visit it can readily do so by carriage.

The lesson of the failure of the South Fork Dam is undoubtedly a too contracted spill-way; while it is true that the rainfall which caused

Digitized by Google

it was unprecedented; this cause has been made more prominent by the failure of the Walnut Grove Dam. This dam, which was for the purpose of irrigation, was located in Arizona. The dam was over 400 feet long on top, less than 100 feet at the bottom and about 110 feet in height. The rain which caused its destruction was, like that of the South Fork disaster, unprecedented in volume, which only goes to show that provision must be made for greater than any recorded flow of water.

Antiquity of the Engineering Profession.

At the annual dinner of the Boston Society of Civil Engineers, early in 1889, at which the writer represented this Society, I was struck with the remark made by President Eliot, of Harvard, that the profession of the engineer is the oldest known to history. A little investigation shows this to be true, as long before either law or medicine became a science, and when the priesthood was but a guild of charlatans, man was obliged to wrestle with the facts of nature and with the conditions in which he found material things. We have long been accustomed to refer to the Roman aqueducts as specimens of ancient engineering, and they are certainly worthy of consideration. A treatise by Consul Sextus Julius Fontinius, written about A.D. 50, stated that there were then aqueducts bringing water to the City of Rome with an aggregate capacity of 28 000 000 cubic feet per day. One of these aqueducts was 63 miles long, 6½ miles of which were on arches, some of them over 100 feet high. The aqueduct which supplied ancient Carthage is stated to have been 70 miles long and to have had one arcade consisting of 1 000 arches, some of them 100 feet high. It is stated, however, that aqueducts in Peru, constructed in the time of the Incas, far exceeded in length or capacity any other aqueducts of ancient or modern times, some of them being constructed to bring water for irrigation from the mountains several hundred miles off. The practice of medicine probably first received the character of a science from the time of Hippocrates, who was born in 460 B.C., for although physicians are occasionally referred to as disciples of Esculapius, he is believed to have been rather of the nature of what would now be termed a quack. The Roman civil law, to which all our laws refer, was first codified by the Emperor Justinian, and as Rome was founded in 535 B.C., Roman law cannot go back to an earlier date. Recent explorations in Egypt, a description of which is given by Miss Amelia B. Edwards, in an article upon the ancient Egyptian City of

Digitized by Google

Bubastis, show that in the time of Mena, about 5 000 B.C., a dike or dam was constructed, which turned the course of the whole River Nile into a bed several miles at the most distant point from its former location, thus showing that at that remote period engineering works were undertaken of a magnitude which finds no parallel in modern times.

Degree of Civil Engineer.

The term "Engineer" is, unfortunately, in this country a very misleading one. In addition to the engineer referred to by our Past President, William E. Worthen, whose professional title he derives from "*Le Genie*," genius, and going still further back, with that gallantry which we all recognize in Past President Worthen as co-equal with his talent, he derives the latter from the Greek *gune*, woman. Certain it is that without woman there would be no engineers, because without woman the human race would have continued in a savage state with no occasion for engineers or engineering—but this is a digression; what I designed to say, was, that we apply the term engineer alike to the man who designs and to the man who operates the engine, and worse than that, we use the term "Civil Engineer" as a degree conferred by colleges and engineering schools, as unmeaning in its application in many instances as the LL.D. formerly bestowed by Harvard without discrimination upon the Governors of Massachusetts.

It is gratifying to know that some of the schools (notably the Institute of Technology of Boston) have reformed in this respect and are conferring the degree of S.B. (Bachelor of Science) alone on graduates from any of their regular courses.

The degree of civil engineer is one which is not properly conferrable as a result of college studies alone, and the title should not be applicable to any persons who have not had a certain amount of experience and practice in the designing and construction of engineering work; in fact, the qualifications required for membership in this Society should be the qualifications entitling any one to use the title "Civil Engineer." I desire in this connection to repeat what I have previously stated in a paper read before the Society, that the curriculum of our engineering schools should embrace the science of accounts and the study of the law of contracts, to such an extent as to enable a young engineer to keep proper accounts of his expenditures and to know when he was transgressing the well-known principles of law. The importance of a

Digitized by Google

knowledge of accounts goes without saying, and the desirability of at least a general knowledge of the law of contracts is easily illustrated by an occurrence within my knowledge.

The president of a railroad company made a contract for the timber and lumber for the construction of extensive shops, providing for delivery on board cars at the saw-mill, when a branch road should be opened to that point. When the road was opened the material was loaded on cars and forwarded to the location of the shops. The engineer in charge of the construction of the shops inspected the timber, rejected a very large portion of it as not up to the standard of specifications, and notified the contractor that it was held subject to his order. The engineer did not even take the precaution to measure the timber which he rejected, or to take any account of it. The result was that in due time the contractor brought suit for the value of all the timber which he had loaded on the cars, and by making his proof by his own employees that he had loaded certain timber on the cars, the company found itself estopped from introducing the testimony of the engineer as to the character of the timber received at the shop location, for the reason that the contract specified delivery on cars at the mill, and under the law of contracts the inspection and rejection, if any, must take place at that point. The mistake cost the company several thousand dollars, for which, of course, there was no corresponding benefit.

Engineers are frequently so situated that, like the captain of a ship, they must, for the time being, be a law unto themselves.

American Engineers in Europe.

One of the notable events of the past year was the visit to Europe of the American Engineers, composed of members of the American Society of Civil Engineers, the American Society of Mechanical Engineers, and the American Institute of Mining Engineers. The party, which included fifty-two members of this Society and thirty-three ladies accompanying them, left New York during the last week of May and first week of June, and met with a most courteous and enthusiastic reception from the Institution of Civil Engineers of Great Britain, the Institute of Mechanical Engineers, and the Iron and Steel Institute of Great Britain, the Society of Civil Engineers of France, and quite a number of corporations and individuals throughout Great Britain, France and Germany. The most notable courtesy shown to the party was probably the dinner given by

Digitized by Google

the Institution of Civil Engineers in the Guildhall of the City of London, by the express sanction of the city authorities. This marked courtesy was particularly striking, inasmuch as a similar courtesy had been extended but once before to any non-municipal visiting party to London. While the excursion was not in any sense a Society matter, every member of the Society must feel a gratification in the fact that its members were so cordially received by their brethren and by notable personages abroad. It may be stated in this connection that a large party composed of members of the Iron and Steel Institute of Great Britain and the kindred societies of Germany, will visit this country in October next, and this Society has been cordially invited to co-operate in their entertainment. Your principal executive officers have been given prominent places on the Committee of Arrangements, and such of your members as are able to do so are cordially invited by the Committee to participate in a welcome to our foreign guests.

These and similar courtesies tend greatly to improve the standing of the profession, and thereby indirectly benefit its members.

Expert Testimony.

I take this opportunity to indorse, from a purely professional point of view, and also from a regard for the personal interest of the members of our profession, what was so well and wisely said by Past President Worthen in regard to the testimony of experts.

Nothing can be more unseemly than to see two eminent members of any profession each striving to earn his fee by stating such facts or giving such opinions only as make for the side by which he is retained. The differences of opinion which naturally must exist are thereby stimulated and intensified until all respect for scientific opinions is lost by the laymen, who have no knowledge of the subject.

It has long been my opinion that the expert witness should be called by the Court, and that he should be allowed to state facts and opinions, freed from the trammels of the *suppressio veri* so often employed by the attorneys on either side. It might result in some of my professional brethren receiving less fees, but it would be to the benefit of the profession and the cause of justice.

Digitized by Google

Necrology.

Our necrological record during the past year is particularly sad. We mourn the loss of three Past-Presidents, two of them Honorary Members; eight Members, three Fellows and one Subscriber.

To many of us, the faces of our Honorary Members, Horatio Allen and William J. McAlpine, had become familiar as among our earliest recollections of the Society, while in the loss of Past-President Fred. Graff and Major Otho E. Michaelis, the Society has sustained losses which cannot be replaced.

I cannot close this address, without again calling the attention of our membership to the importance of care and accuracy in all their acts and statements. The engineer, of all men, cannot afford to be careless, to make loose or unauthorized statements, to give opinions for which he has no data, or to do any act even of so unimportant a nature as sending in a ballot to this Society, without carefully considering and complying with all the necessary forms and being sure of his facts. To the young engineers especially, I would say, guard well your facts, let your estimates be based on known or ascertained data, not guesses based upon nothing. Great odium has been brought upon the profession from the fact that, from one cause or another, the estimate of the engineer is too generally below the resulting cost, often far below, until capitalists when shown the anticipated cost of a work in which they are asked to invest, will often say, "Oh, that is only an engineer's estimate."

It is on record that the actual cost of the original Croton Aqueduct, when completed for use, was less than 1 per cent. in excess of the original estimate made by that careful engineer, our late esteemed Member, John B. Jervis.

By a careful study of all the facts, it is possible to make our estimates bear a much more satisfactory relation to the cost, than, in large works especially, has generally been the case.

This principle should not be confined to matters of costs. In questions of any kind upon which an opinion is to be given or a statement is to be made, let it be based upon the best possible ascertainment of the facts, for the statement of the civil engineer should "import absolute verity."

The Society is growing, its influence is widening, and its power increasing, but whether or not for good, must depend upon the interest taken by its members.

Digitized by Google

The life-blood of the Society is in its papers and discussions. When interesting papers flow in, interest is stimulated, discussions follow, and by the interchange of opinions and experiences, the membership is benefited.

Most engineers have some experiences which are unique—they should be reported. Papers need not necessarily be long, or cover the whole of a subject, and every member should contribute what he can to the general fund of knowledge and experience, and thus the Society can be made to expand in membership, in influence and in value, until it shall embrace all the members of our noble profession on this continent, with a few from the old hemisphere, and it will become in fact, what it has long been in name, THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Digitized by Google

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

439.

(Vol. XXII.—June, 1890.)

A METHOD OF TAKING CROSS-SECTIONS IN DEEP ROCK CUTS BY TRIANGULATION.

By F. W. WATKINS, M. Am. Soc. C. E.

In the construction of the New York, Ontario and Western Railroad, a very heavy cut was required for the approach to the Western Portal of the Weehawken Tunnel.

This cut was nearly half a mile long and increased in depth until at the Portal it was 100 feet deep.

The material excavated varied very much in classification, but was largely in rock of different degrees of hardness, from hard and compact trap to a soft and friable sandstone that would crumble in the fingers.

Under the contract, the material was classified into earth, hardpan, loose rock (that could be removed with a pick without blasting), solid rock (that required blasting), and the depth and classification of each stratum could only be determined as the work progressed. The final width of the cut at the surface, and the slope at which the material would stand, could not be determined until hard solid rock was reached.

The character and depth of the cut required that it should be excavated in benches or steps. The work was done by the use of two steam shovels, and the material removed by trains of narrow gauge railroad dump-

Digitized by Google

ing cars. Starting in at the upper levels, making a cut or trench, backing down to make one or more cuts to widen out the excavation, and then going back to start in again at a lower level, and continuing this process until the grade of the road-bed was reached.

This, of course, resulted in a very ragged cut during construction.

On one side of the cut the rock was so soft at several points that it would not stand at the slope proposed, and sloughed off in large slices, and the earth strata above it had to be widened out and trimmed down.

The time required to complete the excavation was about two years, and at its close it was necessary to re-cross section it for a final estimate.

In the deepest part of the cut, the elevations of the top of the sections at the final slope stakes, and of the road-bed at the bottom, were easy to obtain by leveling, but at the "intermediates," on the steep earth and rock slopes, many places could only be reached by ladders, or by lowering a man with a rope, which, it being the winter season, was attended with some danger.

Triangulation by horizontal and vertical angles was then resorted to, and the cross-sections were taken with ease and accuracy.

From four to six sections were to be taken in each 100 feet. The center line of the double track road-bed was taken for a base line, and centers and grade pegs carefully marked for each station and section.

A transit with a vertical circle and horizontal level tube attached to the telescope was then set on the center line, from 150 to 200 feet from the cross-section to be taken, and the elevation of the center of the telescope above the grade measured.

Two men were stationed at the slope stakes for the section to be taken, on each side of the cut, and carried between them a long piece of twine, at the middle of which a small bright plum-bob was suspended. This plum-bob was drawn across the cut, lowered and raised to touch each point desired for the section, and the horizontal and vertical angles to that point measured with the transit and recorded in the field-book.

A simple multiplication of the length of the base line for that section into the natural tangents of the horizontal angles, gave the distances out from the center.

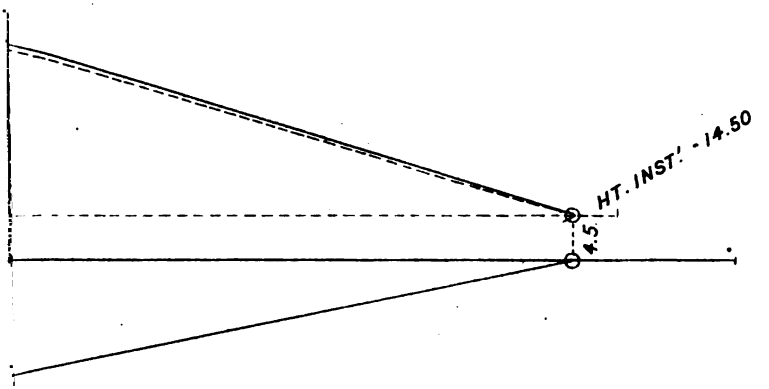
The horizontal base line, for the vertical triangulation, would be the hypotenuse of the right angle triangle of which the center line base and "distance out" were the two sides, or equal to the secant of the horizontal angle for a radius equal to the base.

Digitized by Google

Digitized by Google

PLATE LXV.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXII. Nº 439.
WATKINS ON
CROSS SECTIONS IN ROCK CUTS.

MEASURED.



Digitized by Google

Digitized by Google

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

440.

(Vol. XXII.—June, 1890.)

CALCULATIONS OF THE MEAN HORSE-POWER OF A VARIABLE STREAM AND THE COST OF REPLACING THE POWER LOST BY A PARTIAL DIVERSION OF THE FLOW.

By WILLIAM H. GRANT, M. Am. Soc. C. E.

WITH DISCUSSION.

The City of New York, in diverting a part of the waters of the Bronx River to reinforce the Croton supply, necessarily diminished the natural volume of the stream flowing through the Bronx Valley; from this arose legal questions and claims on the part of property owners affected, and the matter thus came before a Commission which was duly appointed to receive testimony as to alleged grievances and to award compensation in cases where it was found to be due. I was employed on the part of several of the property owners to aid them in presenting their claims before the Commission. Their claims were mostly of a nature requiring some knowledge of hydraulic, sanitary and landscape engineering to develop them and put them in a presentable shape.

I will only in this paper deal with the hydraulic questions; it will not be out of place, however, to say that the claims which arose on sanitary grounds were not without reason, as may be judged from the general

Digitized by Google

fact that a large stream flowing through an alluvial formation, with, in many places, a wide shallow channel, was diminished about one-half in volume, exposing an oozy bed and reducing the cleansing power of the stream, which received a considerable amount of impure drainage. The question of health was of so serious a nature as to justify a searching investigation; also, claims for impairing the landscape features of tastefully improved property, where wide expanses of water, rippling currents and other accessories constituted in the minds of their possessors elements of value, could not be reasonably ignored. If an artistic landscape on canvas, which is at best but an imperfect transcript of nature's handiwork, is impaired in intrinsic value by a blemish, the scene from which it is taken may also be considered to suffer in value by a like casualty.

The hydraulic claims were for diverting and reducing the water power of the stream for mechanical purposes. The following is selected as one of the most prominent cases in which I was employed. The property is situated 7½ miles below the dam by which the diversion of the Bronx was made into the conduit which conveys the water to the City of New York. The entire stream is cut off by the dam and only the water supplied by the drainage of the country below the dam falls into the old channel of the stream. The first questions to determine were, how much water the stream carried at the point 7½ miles below the dam, before its diversion, and how much remained for effective use after the diversion.

The available fall of the stream, through the property of the claimant, in a distance of 4 000 feet, was 10 feet, so that the quantities above mentioned, respectively, being known, the horse-power of the stream, before and after the diversion, and, consequently, the loss of power, could readily be ascertained.

No gaugings of the stream could be obtained; none probably had ever been made of any value; time, of course, could not be afforded to make them. Beyond a few flood marks which were pointed out at dams on the stream and some vague information as to the minimum flow, no direct data were attainable. Under these circumstances it was necessary to look to other sources for the means of solving the problem.

Monthly observations of rain-fall had been made by the Croton Aqueduct Department, at the site of the Bronx Dam, during a period of four years. On comparing these with observations by the same depart-

Digitized by Google

ment, in the Croton basin, for twelve years, the mean results were found to agree very nearly. This was satisfactory information so far. If the flow-off, or collectible water, in the two cases could be made to appear similarly identical, in proportion to their respective drainage areas, then the means sought for were at hand. It was found from the testimony of engineers whose attention had been specially called to the subject, that the characteristics affecting the discharge of rain-fall of the two drainage areas were very similar; this accorded with my own view from a general knowledge of the localities. The centers of the areas are but about 20 miles apart; the geological conditions are mainly the same, and the improved or cleared surfaces are not very unequal. From these circumstances it was believed to be safe to adopt for the Bronx the rain-fall and the flow-off of the Croton basin as observed and officially reported for a period of twelve years. (Report of Commissioner of Public Works, 1882.) In regard to the rain-fall, it may be remarked that the Croton records show an increased amount over that of the Croton basin as we approach tide water, thus:

	Inches.
The mean rain-fall in Croton basin in twelve years is.....	44.6
" at Sing Sing " eleven "	78.6
" at Tarrytown " " "	50.5
" at Kingsbridge " " "	52.7

From this it might be assumed that the rain-fall of the Bronx area (20 miles nearer tide than that of the Croton) would be somewhat in excess of the latter, and would consequently give a greater proportional flow-off. I did not, however, take this into account. It is quite probable that if the observations for the Bronx had been extended through as long a period as those of the Croton the mean result would have been larger.* In view of this, and also of the further probability of a larger percentage of flow-off for the Bronx, consequent upon its greatly reduced area from that of the Croton, I believe the point would not have been over-strained had I used a flow-off a little above that of the Croton mean.

The Croton record shows a mean rain-fall of 44.6 inches, and a flow-off or collectible result at the Croton Dam and entrance to the aqueduct of 51 per cent. of the rain-fall.

* If this is correct it is probably owing to local causes akin to those which contribute to the excessive rain-fall at Sing Sing.

Digitized by Google

The latter data, therefore, govern the following computations, which are condensed into tabular form:

First.—The mean effective horse-power of the Bronx in its normal condition before diversion.

Table No. 1.—This table, covering twelve years of the Croton record, shows, in the columns *a*, the mean theoretical monthly horse-power of the stream at its entrance upon the property of the claimant (in connection with a fall of 10 feet), as deduced from the flow, the latter being obtained from the Croton record by using a proportion which the drainage area of the Bronx bears to that of the Croton. In order to abridge the table the preliminary figures are omitted. Next, it was necessary to determine how the power could be utilized; this could best be done, without much doubt, by the use of turbine water-wheels. The range of the monthly horse-powers, as shown in columns *a*, was so great that a single turbine could not be employed without incurring a serious loss of power; therefore two turbines were selected of different powers—one of 125 horse-power and one of 45 horse-power. By this arrangement, when the flow exceeded 125 horse-power, as it will be seen it does in several instances, the smaller turbine would be brought into requisition to work up the additional power, having previously worked up the smaller horse-powers below 45. The two turbines could not, of course, work up the full power of the variable flow except when the flow corresponded with their separate or combined capacities. The losses of power incurred in this way, as is well known, are unavoidable in a fluctuating stream, and could only be reduced in this case by employing an additional number of turbines, and this was not considered expedient.

In order to facilitate and simplify the calculations, the horse-powers in columns *a* are modified, dropping fractions and otherwise slightly varying, but mainly reducing the amounts, as shown in the transfer of the modified figures to column *b*. The object of this will more clearly appear in the succeeding tables. A small loss of power is incurred (4½ per cent.) by these modifications, which, with some other slight losses, it will be seen further on are mainly neutralized in the calculations, a very small loss only going to the side, avoiding excess in the general result.

Digitized by Google

TABLE No. 1.

Theoretical Horse-power of the Bronx River.

	May.		June.		July.		August.		September.		October.		November.		December.		January.		February.		March.		April.	
	a	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*	*a*	*b*
1870..	52.0	50	22.0	20	14 1	15	13.1	10	10.0	10	12.4	10	17.8	15	17.8	15	115.1	115	124.0	125	99.3	95	126.4	125
1871..	57.7	55	39.4	40	21.3	20	20.9	20	17.4	15	49.0	50	113.8	115	64.0	60	15.5	15	54.8	55	107.3	105	60.1	60
1872..	36.8	35	36.1	35	16.8	15	38.1	35	35.9	35	32.7	25	78.1	75	42.0	40	57.7	55	35.0	35	40.0	40	94.2	95
1873..	227.7	{125 45}	15.9	15	14.6	15	16.1	15	15.0	15	33.5	25	47.4	45	95.6	95	114.3	115	57.5	55	99.9	95	206.1	{125 45}
1874..	88.8	85	27.8	25	26.1	25	21.6	20	16.1	15	21.3	20	18.7	15	26.1	25	119.5	115	87.1	85	88.4	85	105.8	105
1875..	51.4	50	17.0	15	16.3	15	135.8	{125 10}	26 3	25	16.1	15	62.2	60	52.9	50	17.2	15	98.4	95	80.7	75	173.4	{125 25}
1876..	57.9	55	19.8	20	15.9	15	15.0	15	11.3	10	11.1	10	18.7	20	13.9	10	43.5	45	107.7	105	175.6	{125 45}	167.1	{125 40}
1877..	25.2	25	17.6	15	15.2	15	14.6	15	10.0	10	21.5	20	119.9	115	53.1	50	22.9	20	46.3	45	194.6	{125 45}	88.8	85
1878..	45.5	45	42.0	40	18.9	15	19.1	20	53.7	50	23.7	20	53.7	50	199.4	{125 45}	75.5	75	112.9	105	111.0	105	51.4	50
1879..	57.0	55	24.8	25	18.9	20	32.0	25	31.3	25	19.2	20	23.9	25	57.7	60	41.8	40	74.0	75	119.9	115	149.3	{125 25}
1880..	25.9	25	15.7	15	15.5	15	15.2	15	15.4	15	15.2	15	16.3	15	15.4	15	80.3	75	90.3	85	88.6	85	62.2	60
1881..	38.5	35	47.2	45	16.3	15	15.4	15	15.4	15	15.4	15	15.5	15	45.5	45	20.0	20	100.1	95	166.5	{125 40}	53.8	50

Digitized by Google

Table No. 2.—This table consists of the summing up, in column *AA*, of the total number of months in which each grade of horse-power from 10 to 170 in Table No. 1, column *b*, occurred during the twelve years of the Croton record; thus, in Table No. 1, columns *b*, a monthly horse-power of 50 occurs nine times, and the number 9 is placed in column *AA*, Table No. 2, opposite the 50-horse-power, etc. This is a preparatory basis from which the mean yearly horse-power is next to be obtained. The two classes of turbines selected for utilizing the power are designated in their respective positions in the table, and the number of months shown during which each turbine runs alone and works up the available power without the aid of the other; also the number of months during which the two turbines work in concert.

TABLE No. 2.

THEORETICAL HORSE-POWER OF THE BRONX—Continued.

Horse-power.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	April.	*AA*.
					45-HORSE-POWER TURBINE.								
10	.....	.....	.....	1	3	2	.....	1	.....	.....	.....	.....	7
15	.....	4	9	5	5	3	4	2	2	.....	.....	.....	34
20	.....	2	2	8	.....	4	1	.....	2	.....	.....	.....	14
25	2	2	1	1	2	2	1	1	.....	.....	.....	.....	12
35	2	1	.....	1	1	.....	.....	.....	.....	1	.....	.....	6
40	.....	2	.....	.....	.....	.....	.....	1	1	.....	1	.....	5
45	1	1	.....	.....	.....	.....	1	1	1	1	.....	.....	6
					125-HORSE-POWER TURBINE.								
*50	2	.....	.....	.....	1	1	1	2	.....	.....	.....	2	9
*55	3	.....	.....	.....	.....	.....	.....	.....	1	2	.....	.....	6
60	.....	.....	.....	.....	.....	.....	1	2	.....	.....	.....	2	5
75	.....	.....	.....	.....	.....	.....	1	.....	2	1	1	.....	5
85	1	.....	.....	.....	.....	.....	.....	.....	.....	2	2	1	6
95	.....	.....	.....	.....	.....	.....	.....	1	.....	2	2	1	6
105	.....	.....	.....	.....	.....	.....	.....	.....	.....	2	2	1	5
115	.....	.....	.....	.....	.....	.....	2	.....	3	.....	1	.....	6
125	1	.....	.....	1	.....	.....	.....	1	.....	1	3	5	12
					45-HORSE-POWER TURBINE—Assisting.								
10	.....	.....	.....	1	.....	.....	.....	.....	.....	.....	.....	.....	1
25	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2	2
40	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1	1	2
45	1	.....	.....	.....	.....	.....	.....	1	.....	.....	2	1	5

Table No. 3.—This table will explain the continued process. The first column, *b*, gives the several grades of the theoretical horse-power employed in the computations (as in Table No. 2). Columns *f* and *g* give the proportional number of days' duration due to each grade of horse-power in one year; they are obtained by the following proportion: As 144 months are to 365 days, so are 7 months (*AA*, Table No. 2) to 17.74 days, as in column *f*; and for column *g* the proportion is, as 144 months are to 310 days, so are 7 months to 15.07 days, etc. These fig-

* Transferred to 45 horse-power turbine, Table No. 3, as there is a gain of power by it.

Digitized by Google

TABLE No. 3.

"EFFECTIVE" HORSE-POWER OF THE BRONX.

b. Theoretical Horse-power.	*c.* Percentage of Gate.	*d.* Percentage of Horse-power.	*e.* Effective Horse-power, $b \times d$.	*f.* 365 Days—See explanation of Table.	*g.* 310 Working Days—See explanation of Table.	*h.* Total Effective Horse-power, $e \times g$.	
			45-HORSE-POWER TURBINE.				
	$b \div 45$						
10	.22	.31	3.1	17.74	15.07	46.72	
15	.33	.49	7.4	86.18	73.19	541.61	
20	.44	.58	11.6	35.49	30.14	349.62	
25	.55	.65	16.2	30.42	25.83	418.45	
35	.78	.76	26.6	15.21	12.92	343.67	
40	.89	.80	32.0	12.67	10.76	344.32	
45	Full.	.80	36.0	15.21	12.92	465.12	
*45	"	.80	36.0	22.81	19.37	697.32	
*45	"	.80	36.0	15.21	12.92	465.12	
							3 671.95
			125-HORSE-POWER TURBINE.				
	$b \div 125$						
60	.48	.61	36.6	12.67	10.76	393.82	
75	.60	.68	51.0	12.67	10.76	548.76	
85	.68	.72	61.2	15.21	12.92	790.70	
95	.76	.75	71.2	15.21	12.92	919.90	
105	.84	.79	82.9	12.67	10.76	892.00	
115	.92	.80	92.0	15.21	12.92	1 188.64	
125	Full.	.80	100.0	30.42	25.83	2 583.00	
							7 316.82
			45-HORSE-POWER TURBINE—Assisting.				
	$b \div 45$						
15	.33	.49	7.4	2.53	2.15	15.91	
25	.55	.65	16.2	5.07	4.30	69.66	
40	.89	.80	32.0	5.07	4.30	137.60	
45	Full.	.80	36.0	12.67	10.76	387.36	
							610.53
							11 599.30

$$\frac{11\,599.30}{310} = 37.41 \text{ "Effective" Horse-power.}$$

ures having been obtained, they will, when multiplied into the corresponding horse-power, give the total horse-power. The horse-power wanted is the effective horse-power; therefore, this must be ascertained for the multiplier. Column *c* shows the first step in the process of obtaining it; it gives the percentage of "gate" or flow due to the turbine employed, for each grade of theoretical horse-power, column *b*; *i. e.*, taking the 45-horse-power turbine for an example; when it is running with a flow of 45 horse-power it will be at "full gate;" when the flow is reduced to 35-horse-power the gate will be so far closed as to pass only

* See note, Table No. 2.

Digitized by Google

that flow, or instead of being at "full gate," it will be at $\frac{11}{14}$ or 78 per cent. gate, and so on. Next, these percentages are accompanied by losses of power in the turbine, and the corrections for the losses must be applied to obtain the effective power. Column *d* gives the ratio or percentage of effective power due to the several grades of horse-power of column *b*. These ratios have been ascertained by numerous tests of the working of turbine water-wheels by different experimenters. I have adopted very closely those of Mr. Francis, Past President of the Society, as given by him in Vol. XIII of the Transactions of the Society.

These ratios, column *d*, multiplied into the horse-powers of column *b*, give the effective horse-powers of column *e*. Finally, the effective horse-powers of column *e*, multiplied into the corresponding number of days' duration in column *g*, gives the total effective horse-powers of column *h*. Then, the footings of column *h*, 11 599.30, divided by 310, gives the result of 37.41 as the mean effective horse-power for 310 working days of the year.

Second.—The mean effective horse-power of the Bronx after the diversion of 45 per cent. of the normal flow.

The computations which followed were similar to those for the normal flow, and the tabular statements are omitted. The table corresponding with Table No. 1 is the same as that table, with a reduction of 45 per cent. of the quantities. In the table corresponding with Table No. 2, two turbines are employed, one of 30 and one of 70 horse-power. The table corresponding with Table No. 3 gives:

		Horse-power.
For the 30-horse-power turbine.....	223.9 days run and	2 175.48
For the 70-horse-power turbine.....	86.1 days run and	3 821.46
Making totals of.....	310.0 days run and	5 996.94
To the horse-power is added for the 30-horse-power turbine, while assisting (19.5 days).....		340.74
Making in all.....		6 337.68
In consequence, however, of the reduced flow, the 30-horse-power turbine has run (alone) 49.5 days with only 1.2-horse-power (see Diagram B, Plate LXVII), making.....		59.40
I discard this, regarding it as too small to be of practical value. This leaves, deducting 49.5 from 310 days, 260.5 days' run and.....		6 278.28

Then $\frac{6\,278.28}{260.5} = 24.1$ effective horse-power.

Digitized by Google

Reference is made to Diagram B to explain more particularly the latter table.

We have now, for the purpose of ascertaining the amount of power lost (and to be replaced), the total horse-power of the original stream as shown by Table No. 3	11 599.30
And the total horse-power of the stream, after the diversion, as last above	6 278.28
The difference is the amount of horse-power lost	5 321.02

This is to be replaced (see Diagram B) in 237 days, and $\frac{5\,321.02}{237}$ gives 22.5 as the average horse-power operating 237 days in the year, to replace it.

This concludes the calculations as to horse-power.

In order to bring out a practical result, it was necessary to ascertain the money value of the power lost; or more distinctly, what the claimant desired, the cost of utilizing the water-power remaining after the diversion and the cost of the supplemented power required to bring up the standard to his original power.

The following additional calculations were made for this purpose.

The lost power could not be replaced by water, and steam-power was consequently resorted to. An average of 22.5 horse-power for 237 days in a year was wanted, as shown above; the maximum horse-power required was 33, as shown by Diagram B, and a 35-horse-power engine was selected for the duty.

To arrive at a clear result, it will be seen that it was necessary to obtain:

(*a*) The cost of utilizing the original power of the stream;

(*b*) The cost of utilizing the power of the stream remaining after the diversion, and

(*c*) The cost of the supplemented steam power.

Then $b + c - a$ would express the measure of the loss incurred.

As no works had been constructed for utilizing the water-power, a survey of the ground was made upon which to base a proper plan for a dam and other appurtenances, for estimates *a* and *b*. The facilities for the work were very favorable.

Digitized by Google

The following are the estimates:

Estimate *a*, of the cost of utilizing the normal flow of the Bronx River by turbine water wheels, running day and night during 310 days in each year, at 37.4 mean horse-power.

OUTLAY.

Dam, race, turbine chambers, etc.		$7 000 00	
House		400 00	
One 125-horse-power and one 45-horse-power turbine		1 900 00	
Gearing to connect turbines		80 00	
Superintendent constructing work		1 200 00	
Tools for repairs, stove, etc		85 00	
		$10 665 00	
Renewal in twenty years, 45.8 per cent. on $3 500		1 585 50	

OPERATING AND OTHER ANNUAL EXPENSES.

One attendant, 310 days, at $1 50	$465 00		
Oil per day, at 75 cents	232 50		
Fuel in winter	25 00		
Repairs of machinery and other work.	100 00		
Insurance on $3 500	18 00		
Taxes on $9 500	106 00		
	$946 50		
$946 50 capitalized at 6 per cent		$15 775 00	
Total			$28 025 50

Digitized by Google

Estimate *b*, of the cost of utilizing the remaining flow of the Bronx after the diversion of 45 per cent. of the normal flow, by turbine water-wheels, running day and night during 261 days in each year, at 24.1 mean horse-power.

OUTLAY.

Dam and appurtenances as per estimate *a*		$7 000 00
House		350 00
One 70 and one 30-horse-power turbine		2 100 00
Gearing to connect turbines		65 00
Superintendent of construction		1 200 00
Tools for repairs, stove, etc		75 00
		$10 790 00
Renewal in twenty years on $3 600, at 45.3 per cent		1 630 80

OPERATING AND OTHER ANNUAL EXPENSES.

One attendant, 261 days, at $1 50	$391 50	
Oil per day, at 60 cents	156 60	
Fuel in winter	25 00	
Repairs	80 00	
Insurance on $2 600	12 50	
Taxes on $9 900	111 00	
	$776 60	
$776 60 capitalized at 6 per cent		$12 943 33
Total		$25 364 13

Digitized by Google

Estimate *c*, of the cost of steam-power to replace the water-power diverted from the Bronx.

OUTLAY.

One 35-horse-power engine and boiler, setting same, etc	$3 000 00	
Gearing to connect engine and turbine	75 00	
Extension of turbine house	100 00	
Tools, in addition to tools for turbine	60 00	
	$3 235 00	
Renewal in twenty years, at 45.3 per cent	1 465 46	

OPERATING AND OTHER ANNUAL EXPENSES.

Coal, 3½ pounds per horse-power per hour, average horse-power 22½, with 14 per cent. added for friction, equals 26.2 horse-power for 237 days, making 261 tons, at $6	$1 566 00	
Insurance on $3 235	16 00	
Taxes on $3 235	36 00	
Engineer, 237 days, at $3 75	888 75	
Oil and waste, 237 days, at 70 cents	165 90	
Repairs	75 00	
	$2 747 65	
..$2 747 65 capitalized at 6 per cent	$45 794 16	
Total		$50 494 62
Estimate *b*, brought forward	$25 364 13	
Estimate *c*, brought forward	50 494 62	
b + *c*	$75 858 75	
Estimate *a*, brought forward	28 025 50	
b + *c* — *a*		$47 833 25

This is the cost to replace, by water and steam power, the water-power diverted. In other words, the cost of utilizing the undisturbed water-power of the Bronx, deducted from the cost of restoring and utilizing the power, after the diversion, leaves the sum of $47 833 25 as the cost to the proprietor of the property for making good his loss of power.

Digitized by Google

General Remarks.

From the nature of the case, the first branch of this subject—that of the horse-power of a variable stream—admits of only an approximate solution; the nearest approach which could be made to accuracy of result would have been by the use of an extended series of gaugings of the stream, which were not attainable. The measurement of flowing streams is an old subject, which has been investigated with great ability and labor by hydraulicians, for many years and for many valuable purposes, but the best processes and the accuracy of results are still mooted questions. The case presented here was to obtain the *nearest approximation to accuracy* as to the horse-power of the stream that was practicable; this, of course, governed the final result reached by the subsequent calculations.

The onus rested upon the claimant to show what the amount of his damage was; if he could not show the exact amount it was incumbent upon him, at least, not to exaggerate it, to keep within reasonably demonstrable bounds. By this course he was liable to lose something of his just dues; but this was the unavoidable possibility of the situation. These considerations were kept in view in the foregoing calculations; and whatever error has occurred is, therefore, believed to be on the side opposed to excess. How nearly the most practicable approximation to accuracy has been made, the data and processes resorted to must speak for themselves.

Digitized by Google

DISCUSSION.

CHARLES E. EMERY, M. Am. Soc. C. E.—The power of the small stream referred to in the paper could not be utilized without the addition of a steam-engine operated in connection with the turbines, and this would very much reduce the value of the water-power. I was engaged on the part of the city of Fall River, Mass., as one of the experts in the legal proceedings initiated to condemn for city uses a portion of the water from the Wautuppa Ponds on the hill back of the city, the flow from which was utilized by a series of mills on the outlet. The condemnation diverted about 67-horse-power. Engines were already required in the mills, and we took the ground on the side of the city that the measure of damages should be on the basis of the cost of the slightly increased steam-power necessary in each mill to make up the water-power diverted, the mill-owners being allowed a certain amount for repairs and a sum representing a certain proportion of the original cost of the engines. It was urged on the part of the mill-owners that the city should put small engines of from 7 to 10-horse-power in the several mills to replace the power. As the cost for horse-power is considerably greater for small engines, the damages calculated on the basis that they proposed were considerably larger than on that urged by the representatives of the city. There was, however, one case, a print works, where the water was not used for power, in respect to which it was urged that the value should be calculated on the basis that it could only be restored from the water mains of the city. The experts adopted substantially the measure of damages proposed by the city in relation to water used for power, but allowed damages to the print works at a much higher rate. The amount of coal per horse-power named for a 30-horse-power engine in the paper, viz., 3½ pounds per horse-power per hour, would require a better engine than can be obtained in the market. A fair estimate of cost for the various sized engines, taken substantially from my testimony in the Fall River case, will be found in my paper on "The Cost of Steam-Power," *Transactions* Am. Soc. C. E., Vol. XII, page 425, November, 1883.

A MEMBER.—What were the damages in that case?

Mr. EMERY.—On the part of the city we figured the damages at $35 510 for 66.4-horse-power. It was made up on the basis of an allowance to the owners of the several mills of a certain proportion of the actual cost of the steam machinery in the various mills required to keep the power uniform the year round, notwithstanding the variation in the water supply, and an allowance of the capitalized cost of the power diverted on each of the several falls with steam machinery in use of the sizes above referred to.

The average flow in the stream would produce about 1 340-horse-power. It was calculated that a mill requiring an average of 623-horse-

Digitized by Google

power needed steam machinery of 550-horse-power in order to continue its work the year round. The water diverted in this case would only produce 7-horse-power for the fall at this mill, and the award was based on the cost of producing that much power in a 550-horse-power engine. In another case, however, of a small mill where an engine of only 115-horse-power was required, the water diverted would produce 8-horse-power on that fall, which had to be settled on the basis of the cost of the power in a 115-horse-power engine. The paper referred to gives the cost of the horse-power for the different sizes of engines from 400-horse-power down to 5-horse-power. The present value of each horse-power, including cost of machinery and buildings and an amount of capital which, invested, would produce a sufficient amount for continuous operation indefinitely, including renewals, is stated at $660 for the 500-horse-power engine and $3 159 for the 5-horse-power engine.

The CHAIR (Past-President WILLIAM E. WORTHEN).—This was power in this case, but the case referred to in Mr. Grant's paper is very different. This stream had never been used for water-power.

Mr. EMERY.—In the case discussed in Mr. Grant's paper an engine should be provided, for the reason that no manufacturing could be done if it entirely depended on such a variable water-power. The calculation should be made on a practical basis, and when an engine is necessary it very much reduces the relative value of the water-power.

Mr. GRANT.—What are your figures for an engine of 35-horse-power?

MR. EMERY.—The tables give 3.27 pounds of coal per horse-power per hour, for a 50-horse-power engine, and 4 pounds for a 25-horse-power engine.

Mr. GRANT.—This is not an estimate of the commercial value of the horse-power; it is simply the cost of utilizing the water-power in two cases, and substituting steam to make up the deficiency.

EDWARD P. NORTH, M. Am. Soc. C. E.—Referring to the two diagrams, they show a very unequal amount of power during the different months of the year. I would like to ask if a mill site is worth more as a water-power than its lowest run? Where it goes up to 150 horse-power occasionally, that is of no value, because the mill cannot be worked to advantage with say more than 18-horse-power. The point I want to make is that a mill site is worth no more than its lowest horse-power.

Mr. GRANT.—This paper has nothing to do with the marketable value of the horse-power. It simply deals with the question whether the power exists, and it is practicable to use turbines in the way suggested to utilize it.

I will state that, in this case, an expert who was familiar with the value of water and steam power and experienced in the purchase, sale and leasing of water-powers, estimated on the basis of commercial value, a considerably larger sum for damage to the claimant than my figures for replacing the power diverted.

Digitized by Google

Mr. EMERY.—Mr. North's inquiry brings up the whole subject of utilizing water-power in a variable stream. He puts the question whether or not the value of the power should not be based on the minimum which will be obtained under the most unfavorable circumstances. That is very nearly true when manufacturers have to depend upon the water alone. Generally only a small portion of the work can be postponed till there is plenty of water. In all manufacturing centers, water privileges are now supplemented by steam-power, and, in many cases, sufficient steam-power is provided for the total amount of work to be done, and water is used simply to reduce the coal bills. The modern automatic engines will automatically adjust the amount of steam used to that required to maintain the power, and the quantity of coal burned will be reduced proportionally. Such prices apply only where the power has been utilized. In other cases, it would seem that the valuation should depend more upon real estate rules and customs than engineering principles.

Mr. WORTHEN.—Could you run a 25-horse-power engine and a wheel intended for a 30-horse-power, with only 4-horse-power on it, together with advantage? Would not the engine run the turbine?

Mr. EMERY.—It would be better to shut the turbine off.

Mr. GRANT.—Is that not a very extreme case?

Mr. EMERY.—Yes, but it is well to bring up the fact intimated by the Chair, that turbines are of inferior economy when run with a fraction of their power. Wheels have been recently made which can run at half-gate with much better results than heretofore, but when the power is still further reduced, the wheels are necessarily very uneconomical.

JOSEPH P. DAVIS, M. Am. Soc. C. E.—I would ask, Mr. President, whether it is proper to discuss this subject at this time? There are members present, some engaged on one side in this suit for damages and some on the other; therefore, to me, it does not seem wise to discuss the subject here.

If I understand the paper, it is not theoretical simply, but is intended also as an estimate of damages.

From it the natural conclusion is, although I believe it is not so stated, that the damage to the property is $47 000.

Now the true measure of the damage is simply the difference between the salable value of the property before and after the diversion of the water. The water-power has existed there unused ever since nature made it, and probably it has never had a salable value of $2 500. In arriving at results you must consider to what profitable use the power can be put after it is developed. A water-power in the Rocky Mountains would have a small or no salable value, but if you were to estimate its value by the cost of replacing it by steam-power you would get results even larger, per horse-power, than on the Bronx. As before stated, the real measure of the damage is simply the reduction in the salable value of the property by the diversion of the water.

Digitized by Google

It strikes me that in the calculations of this paper there has been a too general use of averages. For instance, a diagram presented shows a very large horse-power, lasting a few days, but how it can be utilized I do not see, except, perhaps, as Mr. Emery suggests, by supplementing at other times with steam-power.

Mr. GRANT.—This discussion seems to me to have diverged from the subject. The paper presented here relates—as its title implies—to the mode of determining the mean horse-power that exists in a variable stream with a given fall, and the cost of replacing such portion of the power as may, from any cause, be diverted.

The mean horse-power is, of course, made up of the extremes, the minimum, intermediate and maximum horse-powers.

The uses to which the varying horse-powers may be put do not enter into the subject. For some manufacturing purposes a variable power would not be well adapted, while there are many other cases where a variable power can be used to good advantage.

In the case of a portion of the normal power of the stream having been diverted or withdrawn, an estimate is made, as a part of the paper, of the cost of replacing the diverted power, that is, the cost of adding to the water-power remaining after the diversion a compensating amount of steam-power. This is all that is attempted by the present paper. The questions are wholly independent of the local case which they happen to be applied to.

I note, briefly, the points raised by members during the discussion, so far as they are not covered by the preceding explanation.

A member asked "What was the measure of damages?" I replied $47 833. It would have been more exact if I had said this amount is the cost of replacing the diverted water-power.

Mr. North inquired "What the assumed value of the horse-power was before the diversion?"

The Chair replied: "There was no diversion; it was unimproved water-power."

I do not understand this reply.

Mr. Emery stated "That the water-power of the (normal) stream could never be utilized without the aid of a steam-engine." I think this needs qualification, in accordance with what I have remarked above.

He thought my estimate of 3½ pounds of coal per horse-power per hour for a 35-horse-power engine was too low, but upon referring to a paper which he formerly presented to the Society, I believe he found he was incorrect.

Mr. Davis' remarks, in part, relate more to supposed legal points in the estimate of damages than to the topics of the paper; as to his comments on the impracticability of utilizing a variable flow, I must refer to my preceding explanation. As this point does not strictly come within the subject of the paper, I avoid enlarging upon it.

Digitized by Google

If any errors are pointed out in my calculations or data I shall be glad to explain or correct them.

Mr. North notices "that the diagrams show a very unequal amount of power during the different months of the year, and makes the point that a mill site is worth no more than its lowest horse-power." This view emphasizes some of those previously expressed. I believe it needs qualifying very much, but for the reason before stated I avoid enlarging upon it.

I wish to say in reference to the query the Chair addressed to Mr. Emery during the discussion, as before stated, "Could you run a 25-horse-power engine and a wheel intended for a 30-horse-power, with only 4-horse-power on it, together with advantage? Would not the engine run the turbine?"—that I do not find in my paper anything upon which reasonably to base these amusing stumbling-blocks.

In my reply to the discussion which I presented February 24, 1887, I covered all the principal points that were raised, according to the notes which I took of the matter at the time.

The report submitted to me in proof of Mr. Charles E. Emery's remarks contains some things which I must have failed to make note of, and it will be proper for me to make further reply to them.

First.—As to the practicability of utilizing the water-power of a variable stream without the aid of a steam-engine:

It is doubtless desirable, in general, to have a co-operating engine, but, as I have previously remarked, there are many cases where a variable water-power may be usefully utilized without one. I might cite instances in confirmation of this, but it does not seem necessary to take up time and space for it.

Second.—His remarks in reference to his investigations of water and steam-power at Fall River, Mass., contain some useful information, but I am at a loss to understand how they affect the strictly construed questions treated of in my paper.

A general inference seems to have been drawn, upon a hasty glance at the subject, that the object of the paper was to exhibit the merits of local claims for compensation for diverting water-power. I admit that this was the origin of my investigations. I would have preferred, in submitting them to the Society, to have eliminated all reference to the local case, but it was impracticable.

I may be permitted to say, that there was no intention to influence opinions in regard to the local questions, and that I believed that the paper, as presented, was an abstract matter, to be considered strictly (as its title implies) as illustrating a mode of treating, in general, a subject of hydraulic interest. Hence, it appears to me that the introduction of arguments in the discussion, outside of that view, has a misleading tendency, obscuring the subject, rather than supporting or refuting it.

Digitized by Google

Third.—As to the amount of coal per horse-power per hour for a 35-horse-power engine, I refer, for brevity, to the statement I previously made during the discussion when Mr. Emery thought my estimate was too low, that upon his referring to his paper he found that he was mistaken. The paper is contained in the Transactions of the Society, which I examined, and found there was no material difference between his figures and my estimate on the coal question.

The Chair remarked in reference to Mr. Emery's allusion to a horse-power of water in the Fall River case, which he had investigated and which had been utilized: "This was power in this case, but the case referred to in Mr. Grant's paper is very different; this stream had never been used for water-power."

The Chair had previously stated, during the discussion, in reply to a question by Mr. North as to the value of the horse-power of the stream before the diversion, "there was no diversion; it was unimproved water-power." I thought at that time that I might have misunderstood him; it seems not, however. In his view, then, if correctly reported, no water-power existed and none could be diverted from the stream because the power had not previously been improved or utilized.

It appears to me, with great respect for the opinions of the Chair, that if the law of gravitation is not suspended there is a power in any flowing stream of water, having a given fall, whether such power is usefully utilized by turning a wheel or is permitted to run to waste. It would be as unjust to the proprietor of such a power to deny it as it would be to say to the owner of a gold mine that his gold did not exist because he had delayed to dig it out and obtain the market value for it; that he could have no claim for reparation if another party took possession of it or deprived him of the privilege of utilizing it.

As the Chair remarked during the discussion that he differed largely from my views, and also that he was retained as counsel or expert in the Bronx case, it will not be out of place to mention that in his testimony in that case he differed from me as to a fact which I had assumed (but had carefully examined), that 51 per cent. of rain-fall had been found collectible from the Croton watershed after twelve years' daily observations by the Croton Aqueduct Department, and that such percentage could be properly applied to the Bronx region as a collectible or utilizable quantity for power purposes. He considered 51 per cent. as "beyond all precedent."

This simply raises the question:

1st. Whether the Croton observations are or are not correct; and,

2d. Whether they are applicable to the Bronx region?

The first has been so very generally adopted by engineers as correct and reliable, and has been so well corroborated by observations in similar localities, that I can adduce nothing further to strengthen it.

Digitized by Google

The second is a matter to some extent of judgment, and there may reasonably be some difference of opinion in regard to it.

I have stated, heretofore, the grounds upon which I assumed it, and I trust they will generally be considered as tenable.

I will only remark further upon a point which arose during the discussion and which is only indirectly relevant to the subject:

The Chair suggested that the discussion, when the paper was read, be postponed until later, as there were some Bronx cases still pending, though later in the discussion he thought it might properly be considered as a purely engineering question.

Mr. Davis thought the discussion should be postponed, as several members were connected with the Bronx cases.

The Secretary kindly informed me, in explanation of the delay in publishing the paper, that objections had been made to its publication by members of the Society on the ground that the subject was in litigation in the courts, and that several members were retained on one side or the other as experts or witnesses.

I explained at the time of the discussion, as these points came up, that it was nearly two years since I had been connected with the matter, and that I had since been absent from the city, and understood that the testimony had been closed; that I regarded the matter as merely an engineering question outside of its local references.

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

441.

(Vol. XXII.—June, 1890.)

OBSERVATIONS ON THE FORTH BRIDGE.

By CHARLES E. EMERY, Ph.D., M. Am. Soc. C. E.

It is proposed in this paper to state in a conversational way the information obtained in connection with a recent visit to the Forth Bridge, with some kindred reminiscences which, it is hoped, will be interesting to those whose engagements did not permit them to visit Europe last summer. My visit was made on the 14th of August, 1889, long after the unequaled excursions and receptions given in England to the engineers from the United States, and after most of the visitors singly or in small companies had seen this great work. Let those present, in imagination, keep me company from Edinburgh, with its evidences of military strength and literary culture, and as we pass over one of those perfect macadamized roads, so familiar abroad, through villages embowered in foliage and lovely expanses of rolling country, with numerous patches of woodland carefully trimmed and freed from underbrush in the unequaled manner found in the English parks, we may fitly examine why the construction of such a bridge was warranted in this location and bring to mind the principal dimensions even if, to interest the few, we briefly refer to matters familiar to the many. This bridge is situated about 8 miles westerly from Edinburgh in 56 degrees N.

Digitized by Google

latitude, corresponding closely with that of Davis Inlet on the eastern coast of Labrador, which is much nearer Greenland than to St. Johns, Newfoundland. The temperature is greatly modified by the bounteous stores of heat sent from our own coast, but the change in the position of the isothermal curves does not make a pleasant climate at this high latitude, as the heat can only be utilized through atmospheric disturbances of greater or less intensity, and constantly recurring rains and mists which transfer the heat from the water to the land. The moisture keeps all the British Isles covered with a mantle of living green, but agriculture is much limited, particularly at the north, where only the trees, grasses and more hardy cereals flourish, and plants as common as the ordinary tomato are hot-house luxuries. Still there is plenty of business. The fishing and woolen industries, the manufacturing interests, the building materials available and the never ending tide of sportsmen, tourist and summer residents make it necessary to run railway trains of considerable length, and at speeds comparing well with those of more favored latitudes. The peculiar formation of the coast, with its deep indentations, would make a coast line, avoiding the hills, a very crooked one, so that years ago it was found necessary, some 40 miles north of Edinburgh, to build a bridge across the Tay, the destruction of which is still fresh in memory. This estuary is now spanned by a substantial monumental bridge, the longest in the world, of which we had the pleasure of receiving very complete monographs from Mr. Crawford N. Barlow, associated with his father on the work. The Firth of Forth prevents direct land communication from Edinburgh to the north, thus giving the North Western and Caledonian systems on the west a slight advantage in distance, which caused such a diversion of the north of Scotland business that the North British Railway, starting from Edinburgh and connecting with the Midland and east side lines of England, felt the necessity of bridging the Forth to obtain its share of the travel. The original enterprise fell flat at the time of the destruction of the Tay bridge, but the commercial demands were so great that new investigations were made, the enterprise was revived and the bridge is now nearly completed. The distance which can be saved by the bridge in traveling from Edinburgh to the important town of Perth at the head of the Forth of Tay is only 12 miles by the existing lines, which are, however, being improved. It would only be 27 miles by an air line, which is not practicable. The new line will, however,

Digitized by Google

have a further advantange of 22 miles in reaching points on the east coast, such as Dundee and Aberdeen.

In this connection let us examine a reproduction of a view published in connection with one of Mr. Baker's papers before the British Association (Plate LXVIII):

"The total length of the viaduct will be 8 296 feet, or nearly $1\frac{5}{8}$ miles; and there are two spans of 1 710 feet, two of 680 feet, fifteen of 168 feet girders; four of 57 feet, and three of 25 feet being masonry arches."*

" The clear headway for navigation will be not less than 150 feet for 500 feet in the center of the 1 710 feet spans. The extreme height of the structure is 361 feet above, and the extreme depth of foundations 91 feet below the level of high water."

" There will be about 53 000 tons of steel in the superstructure of the viaduct, and about 140 000 cubic yards of masonry and concrete in the foundations and piers.

"The wind-pressure provided for is a pressure of 56 pounds per square foot." The total wind-pressure on the main spans is " estimated at nearly 8 000 tons."

The material used throughout is open hearth steel. " That used for parts subject to tension is specified to withstand a tensile stress of 30 to 33 tons to the square inch, with an elongation in 8 inches of not less than 20 per cent. That subject to compression only, a tensile stress of 34 to 37 tons per square inch, with an elongation in 8 inches of not less than 17 per cent." " The tensile strength of the rivet steel is 26 to 30 tons per square inch." The effective depth of the cantilevers over the piers is 330 feet, and at the ends 35 feet. The intermediate spans supported by the cantilevers are 350 feet in length, 50 feet deep in the center, and 37 feet deep at the ends.

The original contract price for the bridge completed was £1 600 000, say $8 000 000, but the cost will undoubtedly be greater, as after the difficulties of the work had been fully developed, an arrangement was made for its further continuance on a percentage basis.

To aid in resisting the enormous wind strains, the cantilevers are widened at the piers, making the general appearance of the converging lines in plan much the same as in a suspension bridge. A distant

* The dimensions are from the paper of Mr. F. E. Cooper, *Jour. Iron and Steel Inst.*, 1888.

Digitized by Google

view of the bridge so much resembles the miniature picture in Plate LXVIII that its size is not for the moment realized, but as we approach closer, and the ground recedes to the Forth and the great bridge towers rise before us, though as yet far away, we feel the same awe as in viewing one of nature's monuments, and the magnitude and well adapted proportions of the structure immediately claim attention. We had been preceded by so many visitors that it was thought best to accept the general invitation without notifying the engineers in London or trespassing specially on the time of anybody connected with the work; so alighting at the Queensferry office, a simple personal card, sent in without directions, was taken to the contractor's department, and the customary old guide sent out to pilot us about the works. Later, however, Mr. Ernest W. Moir, Assoc. M. Inst. C. E., one of the engineers for the contractors, and who expects to come here soon for Mr. Baker, on the Hudson River tunnel work, was met on the structure and abandoned the immediate charge of the preliminary work in starting out the intermediate span, to conduct the writer personally over the Queensferry cantilever; and afterward, Mr. F. E. Cooper, M. Inst. C. E., representing Mr. Baker, showed us through the engineers' offices and furnished some information which will be presented in this paper.

The shops were almost entirely deserted, but contained numbers of tools, most of them of familiar pattern, of which the great bending presses were special objects of interest. The shops are in temporary sheds situated at the foot of the second slope of the bluff and connected with the shore by an inclined railroad leading out partly on piles to the first main pier. The bridge floor is reached by climbing stairs at the end of the masonry portion of the approach, and as we walk out toward the first great pier we observe the substantial and finished manner in which the road-bed is prepared. On the approach, the general appearance is that of a narrow street with central roadway and narrow sidewalks on each side, all finished with cement to form drainage surfaces, finished as smoothly as we are accustomed to see in interior work. There only the longitudinal stringers to receive the rails show above the drainage surfaces in the center. On the bridge, as illustrated on plan, the stringers are laid in steel troughs of sufficient strength to support the wheels of a derailed train, and there are no guard rails. The covering in the troughs alongside the stringers and on the buckle-plates had

Digitized by Google

not yet been commenced. At that time the great cantilevers were practically complete. The connection of the south cantilever to the anchorage was not finished, though they had commenced building out the intermediate span. The bases of the towers are, however, so large that they do not find it necessary to take any special pains in balancing the weights on either side. The cantilever arms are to be held down to their seats on the anchorages by weights instead of links, so, as was stated, to avoid the slack motion which would be necessary in pin-connected links. As we approach the great central pier, the size and magnificent proportions of the structure force themselves upon us. The impression is not that of walking through a rectangular tube, as in our structures, but, on account of the expansion toward the main piers, rather that of moving toward the base of a horizontal tower. The view of the main members as they succeed one another in perspective, gives no idea of their great size and of the distance they are necessarily separated in making the intersections of so large a truss. The cylindrical main chords and struts necessarily spread out into rectangular shape at the intersections, the webs of the ties are attached to the sides of the rectangular portion, and the laterals and stiffening angles coming in from various directions with numerous laps and rivets, make each intersection, though simple as a whole, a study in itself as a mass of detail, and each is different from the other on account of change of angles and position. As we look aloft to trace the general arrangement, the simplicity of the design as a whole forces itself upon one, and the mind becomes occupied with the necessary details required to put such masses into such definite shapes at such great distances from the points of support and from the water beneath. The system of fitting the various parts together in the yard, and taking off for erection one end as the other was progressing, made the plates come together properly, substantially the same as our larger units, but required the great number of peculiar shaped working stages, derricks and supporting devices that were employed in securing the plates together. In this way, however, members were formed larger than it would seem possible to transport and handle on our system. Passing to the north end of the Queensferry cantilever, we could see the work which was just being started on the intermediate span, and right across what appeared a narrow gap another set of men were working out from the south end of the Inchgarvie cantilever. The ends of the main cantilevers terminate in boxed cavities to receive the ends of

Digitized by Google

the main girders of central span, the bottom of each box being of ample strength to take the shearing strains and the sides, making all parts practically one structure as respects wind strains. The ends of the girders of the central span will simply rest in rocking joints on the seats in one cantilever, but the shearing strains at the other end are collected at the top of the girders in projecting arms, from which they are transferred to the bridge seats on the ends of the bottom chords of the other cantilever, through columns provided with ball joints at each end. The columns will be practically vertical in transverse plan, but their upper ends stand inward to conform to the general tapered shape of the structure. Walking back to the center of the tower, one of the elevators used during the construction quickly takes us to the top, and the whole scene is spread before us. The bridge itself being first in mind, we could not resist the temptation of stepping over the handrail to the very edge of the parapet and following out the top chord of the cantilever as it inclined toward the anchorage, then looking down the great braced columns of the tower to the viaduct 200 feet below, or the same vertical distance as the top of the Cortland street chimneys of the New York Steam Company from the sidewalk, when in attempting to follow the columns still further down to their junction with the lower chords and to the water over 360 feet below, a warning giddiness made it necessary to return to the railed walk and view the scene around. Below, stretched out between rugged banks, the great Firth of Forth; the "Dark Forth" upon whose so-called "sluggish tides" Scott's heroes were made to cross. Above lay the straggling village of Queensferry, and close at hand a substantial pier with a long, paved incline leading into the water, which was doubtless the landing of the old ferry giving name to the place, on which boats with shallow ends actually grounded and delivered their cargoes at the very edge of the water. To the northward we study the shape of the next cantilever and follow it down upon Inchgarvie, with its old castle, strangely indistinct for a distance covered by one bridge span. Observing simply that the connecting road at the north must be cut through a rocky spur and that at the south the bridge would be readily met by an embankment from a cut on the hillside, we start to walk down a construction platform swung just outside and underneath the upper chord of the cantilever to the northward. Here we observe the waves of a real continuous girder formed by the top chord itself. The spans of this chord, between supports, vary from 170

Digitized by Google

feet to 50 feet, making bridges in themselves of no inconsiderable length, which deflect by their own weight sufficiently to be detected by the eye. The system of building the parts on the ground, taking off one end for erection and building on the other, insured the proper original alignment, so in construction it was simply necessary to block the chords up temporarily until the rivet holes became fair, and when in due time the blocks were released, the chord took the shape of a girder, continuous over its various supports at the intersection of the web members. Before reaching the derricks at work on the extension, we make our way to the level of the railway by a series of ladders so high and continuous in themselves as to give a good idea of the size of the structure, even where its height is greatly reduced.

An attractive feature of the engineers' office is the room in which are kept test specimens of the steel used in construction, such as bars, rods, bolts and rivets, twisted and hammered into all sorts of shapes, each with its history and a statement of its physical characteristics preserved as a matter of interest to the profession.

In response to my request to be furnished with a statement for presentation to the Society of any investigations or developments which would supplement the interesting information already given, Mr. Cooper stated that there was not much new at the time, but thought that perhaps some experiments they had been making to show the effects of temperature on the structure would form the basis of a "little talk during an evening." The diagrams herewith presented are copies of those showing the motion of the south end of the southern arm of the south or Queensferry cantilever, obtained automatically from the cantilever arm itself while it was still disconnected from the anchorage and supported entirely on the main pier. The upper series of diagrams, showing the lateral and longitudinal movements of the cantilever arm, were obtained by securing a card horizontally on the end of the cantilever arm and arranging a vertical pencil attached to the anchorage to bear on the same. In this case the words east and west refer to the position of the card on the cantilever arm and not to the motions of the arm. The lower series of diagrams, showing the vertical and lateral movements of the cantilever arm, were obtained by securing a card facing south to a board on the pier and then arranging a pencil, carried by the cantilever, to bear on the card. The words east and west refer as before to the position of the card, but in this case they also show the direction of the

Digitized by Google

movement of the cantilever arm. The diagram showing lateral movements from 6 o'clock on the 25th of June to the same time on the 26th is a typical one. From it we find that the east side of the bridge heats up first on a sunny day, so that while the whole arm expands it moves also a greater distance to the westward or to the right on diagram, thus tracing the inclined line. By 11 o'clock in the morning, however, or say by 10 o'clock in the majority of the other cases, the temperatures of the two sides of the truss become equalized, causing the first movement to the eastward or to the left on diagram, and as the sun moves farther west that side of the truss becomes more heated, carrying the end of the cantilever arm still more to the eastward, but without altering greatly the longitudinal extension. Soon after 4 o'clock there is again an equalization of temperature between the two girders and a general reduction of temperature, whereby the end of cantilever is brought back substantially to its original position during the night, tracing the diagonal line shown. The other diagrams show various modifications, due to differing temperatures and differences in the time the action of the sun takes place; for instance, the diagram from 6 A.M. June 27th, to 6 A.M. of the 28th, shows that the sun did not come out in the morning—not an unusual phenomenon in Scotland—but the temperature gradually rises and the girder expands longitudinally, but shows slight, irregular lateral movements, due, doubtless, to the temperature of air currents and rifts in the clouds, until mid-day, when the action of the sun on the west side is very evident, and the eastern movement shown toward the left end of the diagram is maintained until between 5 and 6 o'clock in the afternoon, when a reverse action and considerable shortening of the arm takes place during the night, the temperature rising above the minimum before 6 o'clock, as shown. The diagram from the 29th of June to the 1st of July is a record of two days superimposed. The vertical movements shown on the other series of diagrams are the resultant of several: first, the direct expansion upwards of the metal columns of the central tower below grade, which is slightly neutralized by any expansion of the stone anchorage pier, the resultant of which would be an upward movement of the cantilever and pencil, whereas the movement at the end due to heating the top chord first would cause the pencil to move downwards. An examination of the curves will show, as should be expected, that at times the whole structure rises and falls with the temperature by the expansion of the lower portion of the tower, but at

Digitized by Google

other times, for instance, on the 27th and 28th of June, the higher temperature of the top chord neutralizes the elevation due to expansion of the tower, and the pencil is carried below the position shown during the night. The extreme delicacy of a lever arm extended out as far as this is shown by the diagram of the 26th and 27th of June, which is imperfect on account of work in progress on the end of the cantilever. Time will not allow of closer analysis of the diagrams, but they are presented for reference, as well as a table showing the record of temperatures at the time. (Plate LXIX.)

It must not be supposed that these diagrams were taken from mere curiosity. The influence of varying temperature has had an important bearing upon the design and construction of the structure from the beginning. Mr. Baker, in his original paper before the British Association, refers to the strains due to unequal heating of the various members and of the different sides of the members themselves, and Mr. Cooper called attention to the records of various experiments made in the same direction on this structure. A sample page from their record book is herewith presented, being a "Report of Temperatures" for the 17th of June, 1887. The temperature as shown by a black bulb thermometer in the sun, varied, as shown, from 69 to 92 degrees in the shade for the twenty-four hours from 9 A.M. of the 16th to 9 A.M. of the 17th, the minimum temperature in the shade being 52 degrees and the maximum 80 degrees. At 4 o'clock in the afternoon, at a temperature of 74 degrees in the shade, the average air temperature inside the horizontal tubes, clear of the steel, was 70 degrees, but vessels filled with water and placed in contact with the tubes showed, as per diagram at the bottom of the sheet, 70 degrees at east side of east tube, 72 degrees at west side of same, 73 degrees at east side of west tube, and 96 degrees at west side of same. The record states that the latter temperature is so high as to appear "doubtful," but that it "is perfectly correct, as the water was like a warm bath." The mean temperature in the sun is at the same time recorded as being only 92 degrees, but it had doubtless been higher during some of the time. The above experiments are not only of great value in determining the conditions to which such structures are subjected, but, as will be seen later, have also an important bearing on the means available for finally connecting the several spans.

The records show that all the details of this remarkable structure were practically settled before the work was commenced. The only

Digitized by Google

change noticeable is in the anchorage of the bases of the piers. Mr. Baker's original intention appears to have been to fasten the bases of each of the four main columns of each tower to the masonry, maintaining their distance by struts under the compression due to the average temperature, and then allowing the changes in temperature to change the strain in the struts. Finally, however, the plan was adopted of providing each column with a saddle with a planed bearing surface 550 square feet in area, arranged to move over oiled surfaces on a corresponding bed plate. One corner of the tower is anchored like a turn-table, so that it can only turn, and motion of the two legs on either side is allowed in lines at right angles to the fixed corner by causing the bed plates to work between guides. The fourth leg is free to move in any direction. The lifting of the legs is prevented by bolts. The motion of the saddles has been about as Mr. Baker predicted in his second paper: they will remain stationary for a considerable period until the changes in temperature are sufficient to produce motion, which then appears to take place with comparative rapidity for a time. The records show, however, on the whole, as should be expected, a contraction in winter and expansion in summer.

The most interesting exhibit made by Mr. Cooper, although he said but little about it, was a sketch of a plan under discussion for safely connecting the intermediate girders at the center in the process of construction. It will be realized that as the central girders are built from the main cantilevers, during the process of construction for the time being the ends of the cantilevers will be at the middle of these girders, and necessarily moving up and down laterally and to and from each other, as shown on diagrams, but with nearly three times the longitudinal differences there shown, and liable to much greater differences during storms or higher ranges of temperature. In completing the bridge these ends must be brought together and united with the full section of metal in the chords required at the center of a girder supported at the ends, and as in doing this the chords, 1 710 feet long, will be connected from pier to pier, it is necessary, in order to prevent them from being strained in extension or compression by changes of temperature, to coincidently cut loose, so far as expansion is concerned, the ends of the 350 foot girder from the cantilevers, or to devise something equivalent. In a model, or small structure, it might be possible with a system of pin connections to insert pins in the top and bottom chords at the center when

Digitized by Google

the cantilevers were expanded, and immediately after extract by main force similar pins connecting the girders with the cantilevers; but in a structure of the size of the Forth Bridge, and a 350 foot span to handle, necessarily made somewhat heavy to withstand the strains of erection, a method must be adopted in which there is a definite time for each and every operation, and which can be safely carried to completion in case of storm or accident to person or apparatus. During the process of building out the half intermediate spans, the top chords are temporarily attached to those of the cantilever by plates bolted on, which are sufficiently thin to flex a little when the strain on same is released. The bottom chord at each end rests against wedges on the cantilevers. When the chords had reached the half length, or nearly so, it was proposed to connect them with plates secured by bolts in slotted holes. It was calculated that sufficient friction could be obtained in this way in the space available to withstand the strains due to the moment even when the span was swung, supported at the ends, if necessary, without riveting. When all was ready the bolts in the slotted holes of the lower chord were to be tightened, just as the temperature changed or a little after that, so that positively no more expansion would take place for the day, when, as the cantilever cooled off, tension would be set up diagonally across the webs of the half girders from their tops at the ends to the extemporized joint at the center, until finally the whole weight of the central span would thus be taken on the three points named, exactly as if it were suspended from an unseen cable passing through these points. Under these circumstances the center of the girder would rise as the temperature decreased, opening necessarily the central joint in the top chord, and it was hoped also that the wedges at the heels of the girders on the cantilevers would be released, but if not, sufficient power was provided to force them out, and for the time being the parts operated upon would be practically two cantilevers, with the two halves of the central span hanging in a catenary between them. Evidently the operation would be completed upon the re-expansion of the cantilevers, so that the ends of the top chord would come together against wedges used to fix the exact length, when, as the expansion continued, the ties at the end of the central span would be loosened and removed and the girder swung, supported at the ends as intended.

The southern intermediate span was safely swung on the 10th of October, and it is now known that friction was not depended upon in

Digitized by Google

connecting the lower chords, though temporary slotted connecting plates were used, arranged so that the slots would prevent overstrain in case the expected expansion was exceeded, and the plates were so adjusted that bolts inserted during the expansion of the girder would bring up at the ends of the slots as the bridge cooled off. It appears from the accounts available that the ends were readily brought into alignment vertically and horizontally, but that the expansion was insufficient to bring the bolt holes fair in the bottom chord. This was accomplished on the sunny side by the aid of hydraulic power, but on the other side it was necessary to elongate the lower chord by burning waste in it saturated with naphtha. The subsequent operations were probably as above stated. Such an operation with such masses was necessarily attended with much anxiety and not a little risk, and its safe accomplishment assures the success of a similar operation on the remaining girder and the early completion of the bridge.

In passing, it occurs to the writer that, with rapid work, by slightly varying the routine, all the changes could be made as parts of one operation. The lengths could be so adjusted by the wedges that the top chords of the girder would be sufficiently compressed during the expansion of the cantilever to release the ties at the ends, when, for the time being, the two parts of the central girder would be upheld by strains diagonally upward from the bottoms of same at their ends to their tops at the center, like a roof truss, and as soon as the central joint of lower chords could be made safe the ties connecting the top ends of girders with cantilevers could be cast off and the bridge swung properly the same evening, instead of waiting for a subsequent expansion as explained. In this case the wedges at ends of lower chord of girder would, of course, be released as the bridge cooled down.

In closing, it is proper to say that opportunities to design and construct great works of this character are only possible where large business interests create a demand and capital is available at low rates of interest. Our American bridge engineers have never refused such opportunities to excel as were born of less favored conditions, as is attested by numerous large structures, many containing features in advance of the times. The skill exhibited in designing, constructing and erecting bridges adapted in detail to our special conditions and limited means, thereby aiding in the development of a great and growing country, is the admiration of engineers throughout the world. The

Digitized by Google

last advance is always, however, the subject of congratulation, in which, we feel assured, all will join. The subject under consideration shows that engineers quail not at works rivaling those of nature in magnitude, and which must be built to resist its destructive forces.

To Sir John Fowler and Benjamin Baker, joint engineers of the Forth Bridge, and all others responsibly associated with the design, superintendence and construction of that great work; who dared, after years of study, to say that such a structure was possible, and whose previous achievements warranted the financial support necessary for such an operation; who studied the winds anxiously in the very gorge to be spanned, so that they could, with certainty, resist its greatest forces; who designed the outlining of the structure so as to use the least amount of material; who designated the material available for its construction and made long series of experiments to determine the safe limitations of strain and methods of manipulation; who in preparing the materials, crushed the very rocks, separating, purifying and shaping their strongest elements with fire; who finally erected in that country of history and poesy the three great guardian giants which we have been examining in detail, thrusting their feet through water and soil, and chaining them to the "bulwarks built by time 'gainst eternity to stand,"* and who stretched out their "rugged arms athwart the sky"† to carry for all time a great railway from height to height across the stormy deep to aid in the further advancement of the comforts and civilization of mankind—we tender our honor and respect. We congratulate you all that your anxious labors are now substantially closed and crowned with the success which careful investigation, sound judgment, patient attention and engineering skill warrant, and we greet you as worthy leading representatives of that great body of men "whose office is to harness the powers of nature and guide them to and fro at will."‡

* Montgomery. † Scott. ‡ The Author, 1885.

Digitized by Google

FORTH BRIDGE WORKS.—*Report Upon Temperature.*

June 17th, 1887.

June 16th.—Temperature taken by Black Bulb Thermometer in open.

TIME OF TAKING.	TEMPERATURE IN SUN.
9.15 A.M.	69 degrees.
10 "	74
10.40 "	78
11 "	82
11.15 "	86
11.40 "	86
12 P.M.	83
12.15 "	84
12.30 "	86
12.45 "	88
1 "	89
1.30 "	92
2 "	92
3 to 5.15 "	92 nearly.

52 degrees minimum in shade after 9 A.M. to 9 A.M. of 17th.

80 degrees maximum in shade after 9 A.M. to 9 A.M. of 17th.

4 P.M. 74 degrees on Inchgarvie in the shadé, near Old Castle.

70 degrees average air, inside both horizontal tubes, thermometer clear of steel.

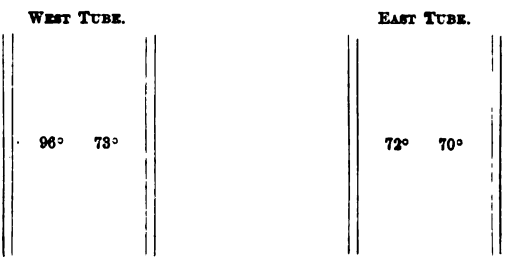
70 degrees east side, east horizontal tube on Garvie, thermometer placed in water next steel plate, inside tube.

72 degrees west side, east tube, as last.

73 degrees east side, west tube, as last.

96 degrees west side, west tube, as last.

This latter being so high may appear doubtful, but it is perfectly correct; the water was like a warm bath.



Digitized by Google

Temperatures of the Air taken in connection with experiments on the Expansion by Heat of the South Arm of the Queensferry Cantilever of the Forth Bridge, during the months of June and July, 1889.

Date. 1889.	Time.	Temperature. Deg. Fahr.	Date. 1889.	Time.	Temperature. Deg. Fahr.
June 24th.	6 A.M.	53	June 27th.	5	67
"	7	53	"	6	68
"	8	53	June 28th.	6 A.M.	54
"	9	54	"	7	54
"	10	54	"	8	56
"	11	55	"	9	59
"	Midday.	57	"	10	62
"	1 P.M.	57	"	11	64
"	2	57	"	Midday.	65
"	3	60	"	1 P.M.	66
"	4	61	"	2	67
"	5	60	"	3	67
June 25th.	6 A.M.	54	"	4	68
"	7	54	"	4.30	68½
"	8	54	"	5	67
"	9	56	June 29th.	6 A.M.	55
"	10	58	"	7	56
"	11	62	"	8	59
"	Midday.	65	"	9	61
"	1 P.M.	67	"	10	61
"	2	69	"	11	61
"	3	67	"	Midday.	62
"	4	66	July 1st.	6 A.M.	54
"	5	65	"	7	55
June 26th.	6 A.M.	54	"	8	56
"	7	56	"	9	58
"	8	58	"	10	61
"	9	62	"	11	64
"	10	67	"	12	65
"	11	71	"	1 P.M.	68
"	Midday.	73	"	2	67
"	1 P.M.	75	"	3	66
"	2	75	"	4	66
"	3	76	"	5	65
"	4	75	July 2d.	6 A.M.	59
"	5	74	"	7	60
June 27th.	6 A.M.	57	"	8	60
"	7	59	"	9	60
"	8	60	"	10	61
"	9	61	"	11	62
"	10	62	"	Midday.	62
"	11	62	"	1 P.M.	62
"	Midday.	63	"	2	62
"	1 P.M.	65	"	3	62
"	2	65	"	4	61
"	3	66	"	5	58
"	4	66	July 3d.	6 A.M.	54

Digitized by Google

DISCUSSION.

T. C. CLARKE, M. Am. Soc. C. E.—The Institution of Civil Engineers, in their late address to American Societies, has well said that one of the great merits of American engineering is the skill with which it has carried out great works with but little money; or words to that effect.

The only instance that I remember where engineers have been unlimited in the amount of money they chose to expend, is that of the New York and Brooklyn Bridge, and that compares very favorably in every respect with the Forth Bridge.

I am a great admirer of the Forth Bridge. I recognize both the architectural and engineering skill shown in its design and execution. I have never ventured to make but one criticism upon it. I said that I thought it could have been built in less time and for less money by the American method of making the parts of such size that they could have been finished at different bridge shops and transported to the banks of the Forth.

The eminent engineers of the Forth Bridge shook their heads at this audacious proposition.

Now I observe, only two years after, that the project of a bridge across the British Channel proposes precisely that method of bringing the parts from a distance and assembling at the site, rather than that of spinning the bridge like a gigantic spider's web.

And the consulting engineers of that scheme are my good friends, Sir John Fowler and Mr. Benjamin Baker, engineers of the Forth Bridge.

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

LOUISVILLE CEMENT.

The undersigned is General Agent for the following Standard Brands of Louisville Cement:

FALLS MILLS (J. Hulme Brand),
BLACK DIAMOND MILLS (River),
SPEED MILLS,
FALLS CITY MILLS,
QUEEN CITY MILLS,
ACORN MILLS,
BLACK DIAMOND MILLS (Railroad),
FERN LEAF MILLS,
EAGLE MILLS.

This Cement has been in general use throughout the West and South since 1830, most of the public works having been constructed with it. Orders for shipment to any part of the country, by rail or water, will receive prompt and careful attention.

Sales for 1888, 1,202,118 Barrels.
Sales for 1889, 1,338,464 Barrels.

WESTERN CEMENT ASSOCIATION,

247 W. Main St., Louisville, Ky.

TRANSACTIONS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

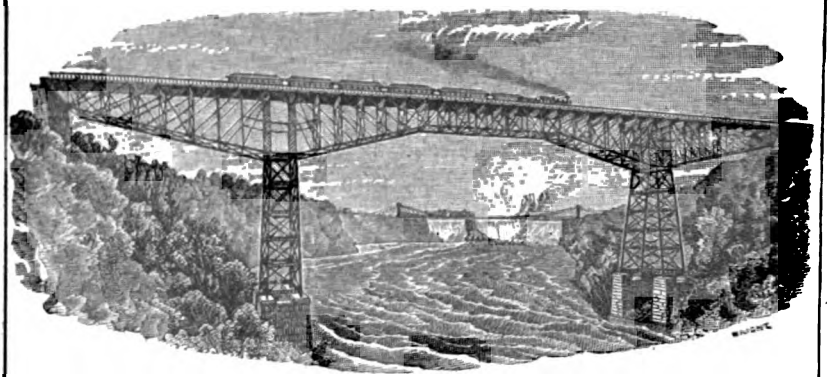
SELECT ADVERTISEMENTS WILL BE RECEIVED AT THE FOLLOWING RATES:

	TWELVE MONTHS.	SIX MONTHS.	THREE MONTHS.	ONE MONTH.
One Page.....	$100.00	$55.00	$30.00	$12.00
One-half Page.....	55.00	30.00	18.00	7.00
One-quarter Page	30.00	18.00	10.00	5.00
One-twelfth "Card".	12.00	.....	.....	....

Address the Secretary of the Society, 127 East 23d Street, New York.

Digitized by Google

UNION BRIDGE CO.



C. MACDONALD, GEO. S. FIELD,

C. S. MAURICE, EDMUND HAYES,

CIVIL ENGINEERS,

And Constructors of Iron and Steel Bridges, Viaducts, Roofs, Elevated Railroads, Marine Piers, etc.

GENERAL OFFICE,

No. 1 BROADWAY, NEW YORK.

Works: ATHENS, Pa.

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google

Digitized by Google